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THE CEMENT EXHIBITS AT THE ST. LOUIS EXHIBITION.

The engineer who remembers the meager exhibits of American Portland cement at the World's Columbian Exhibition at Chicago in 1893, has an agreeable surprise in store on visiting the exhibition now being held in St. Louis. Eleven years ago a Portland cement industry, practically speaking, did not exist in America. These eleven passing years have, however, witnessed a wonderful change. Now the United States leads all countries but Germany in the production of Portland cement. It now produces some 24,000,000 barrels of this important material and this development is naturally reflected in the cement exhibits at St. Louis as compared with those at Chicago in 1893. Another remarkable change brought about in this interval of time is the advent into general structural importance of reinforced concrete. The exhibits of this material at St. Louis are another feature that differentiates the present American exhibition from its predecessor of 1893. It is comparison of this kind that first occur to the engineer who has had the good fortune to study the showing of the two exhibitions in the field of cements and cement products. The exhibition of cement products and of works in cement at St. Louis are, furthermore, of exceeding interest in themselves apart from any reminiscences they may arouse of past and far different conditions. Certainly no engineer who visits St. Louis during the World's Fair period should fail to examine the collective cement exhibit within and surrounding the building of the American Cement Manufacturers' Association.

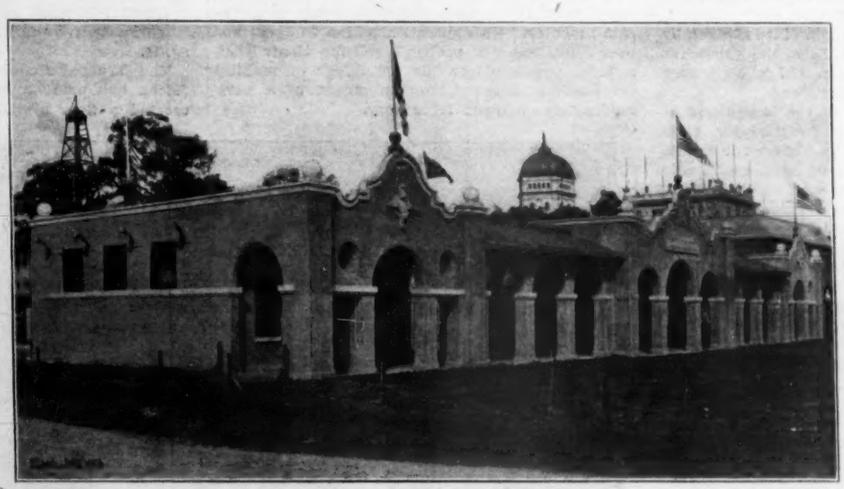


FIG. 1. THE CEMENT BUILDING OF THE AMERICAN ASSOCIATION OF PORTLAND CEMENT MANUFACTURERS AT THE ST. LOUIS EXHIBITION.

SCOPE OF THE EXHIBIT.—The collective cement exhibit at St. Louis embraces:

(1) A collection of the raw materials from which Portland cement is manufactured, together with samples of this material taken in various stages of manufacture to the finished product.

(2) A collection of the various sands, gravels, cinders, broken stone and metal used in concrete, together with photographs and models of structures built of concrete in all parts of the world.

(3) A library of books and files of the various technical journals devoted to cement, mortar and concrete.

(4) A completely equipped model testing laboratory.

(5) A collection of machines for mixing concrete, molding concrete blocks, etc.

(6) A collection of molded and cast structural and ornamental forms in concrete.

The exhibits comprised in groups 5 and 6 are located outside of the cement building, and those of the remaining groups are located inside the building.

THE CEMENT BUILDING. — The cement building in which the larger portion of the exhibits mentioned is housed, is itself an exhibit of the latest development in cement construction, it being built throughout of reinforced concrete. A front view of the building is given by Fig. 1. It consists of three pavilions separated by intermediate courts and connected across the front by a continuous loggia. The superstructure of reinforced concrete rests on a solid foundation footing of concrete having a depth in some places of 16 ft. Each pavilion is one story high, and consists of four walls covered by a double pitch roof carried by beams resting on wall girders carried by the walls and wall columns. The walls are reinforced both horizontally and vertically by 5-16-in. rods, spaced 12 ins. apart. The roof beams are of 30-ft. and 20-ft. span, reinforced by round rods ranging from ⅛-in. to ⅞-in. diameter, and the roof slab is of "ferro-inclave" construction and is 3 ins. thick. The roofing of the loggia is cement tile of a rich red color, and the wall coping and finials are of cast concrete shapes set in place. Ornamental medallions of cast cement molded to represent the labels of the various cement companies are inserted between windows in the court walls. The concrete used for the walls and girders and for the loggia was composed of 1 part Portland cement, 3 parts sand and 6 parts broken stone for the substructure, and of 1 part Portland cement, 2 parts sand and 4 parts chats (screened tailings from the Missouri lead mines) for the superstructure.

The exterior and interior finish of the concrete walls and girders are particularly deserving of notice by engineers. The interior finish of the walls and ceilings consisted in floating the concrete while green with cement mortar and painting the surface. It is the outside wall finish, however, that is of especial interest. Here a rough finish free from cracks and discoloration has been secured, which leaves little to be criticised in its appearance. The mode of securing this finish was as follows: The forms were removed after 24 hours after concreting, and the surface was then scrubbed with soft wire brushes, while at the same time washing the scrubbed surface with a hose stream. The effect of this operation was to remove the cement and sand from the surface, leaving the stone exposed and producing the effect of rough casting.

The three pavilions are utilized as follows: The center pavilion for the model testing laboratory; the south end pavilion for office, reception room and library of cement literature, and the north end pavilion for the exhibits of raw materials in glass jars; the samples of reinforcing bars and mesh, and photographs and diagrams. Perhaps the most interesting exhibit in the north pavilion is that of the raw material and finished cement. The examples have been selected from different sections of the country, so as to represent both the variety in raw materials used for making American Portland cement and the characteristic cement materials of the various localities where cement manufacture is most active. Generally, each exhibit consists of separate jars of each of the raw materials—cement rock, limestone, marl, clay, shale—of a jar of the ground raw mixture, a jar of clinker and a jar of finished cement. There are also jars of coal and of pulverized coal dust for rotary kiln firing. The whole makes a notably interesting exhibit of cement material in its various stages of manufacture to the finished product.

THE MODEL TESTING LABORATORY.—To many the most interesting portion of the collective cement exhibit will be the model testing laboratory, occupying the central pavilion. This laboratory is proclaimed to be the finest that has ever been installed in this country, and a glance at it would gladden the heart of any cement tester. The Cement Manufacturers' Association had a two-fold purpose in installing this laboratory; first, the purpose of exhibiting a laboratory as nearly perfect in its apparatus and appointments as the designer could devise, and, second, the purpose of conducting a series of tests to throw light on certain unsettled questions of cement testing.

The success of the secondary purpose cannot, of course, be disclosed until the results are all collated, but the scheme of work as outlined to a member of the Engineering News editorial staff who visited the collective exhibit in September, by Mr. Richard L. Humphrey, M. Am. Soc. C. E., the engineer in charge, gives promise of useful information. We can, however,

FIG. 2. LEFT INTERIOR VIEW OF MODEL CEMENT-TESTING LABORATORY.

announce now the entire success of the first purpose of the Association. It is so unqualified that we shall give space here to very complete illustrations and descriptions of the laboratory equipment. A right and left view of the laboratory showing all the apparatus in place is given by Figs. 2 and 3. To avoid confusion, the apparatus will first be described, and then will follow a very brief outline of the work being done.

The mixing tables shown in the foreground of Figs. 2 and 3 are specially designed for their purpose. The drawings of Fig. 4 show the details of the construction. As will be seen, each table consists of a raised center portion (one of which carries a moist pan), and two depressed side portions, with glass mixing plates 2 ft. square and 1 in. thick set flush. Two operators can work at one table, using in common the center moist pan. This is 2 ft. square and 6 ins. deep, with a cloth-covered wire screen top, and having a wooden rack in the bottom ¾-in. high. The pan is filled with water to the top of the rack and the cloth is kept wet, thus insuring constant moisture conditions for the test pieces. This moist pan is used solely for storing the test pieces employed for the time of setting tests.

To store the tension and compression test pieces and the soundness pats for the first 24 hours before immersion, there is provided a moist closet constructed as shown by Fig. 5. This is a soapstone box divided into two compartments, each with a planished copper-covered wooden door and cleats for carrying the shelves. The entire inside of the box except the bottom is lined with felt. The bottom shelf is a wooden rack, and each of the upper shelves consists of four glass plates 4 ins. wide and 22 ins. long, each accommodating a gang mold of briquettes.

The test pieces designed for test at greater ages than 24 hours are removed from the moist closet and placed in immersion tanks. These are constructed as shown by the drawings of Fig. 6. There are three soapstone tanks carried by an iron frame. Water is admitted to the tanks at one end and discharged at the other. The drawings show all three tanks the same depth, but as a matter of fact there are two sets of tanks, the topmost tank of one set is 30 ins. deep and the others are 6 ins. deep. It may also be noted that an instantaneous gas heater is attached to the main supply pipe so that the temperature of the water can be kept constant however much its normal temperature may vary.

The equipment for strength tests is unusually complete and interesting. It comprises for tensile tests an automatic, motor-driven Olsen machine and an improved Fairbanks machine. For compression tests there is a special 40,000-lb. machine and a standard Olsen 200,000-lb. machine. The smaller special machine was built by the Falkenau-Sinclair Machine Co., of Philadelphia, Pa., at the request of Mr. Humphrey, and to meet the demand for a cheap and satisfactory compression machine. This special machine will cost not over $300, and will probably sell for about $200, a price which places the possibility of making compression tests within the means of a very moderately endowed laboratory.

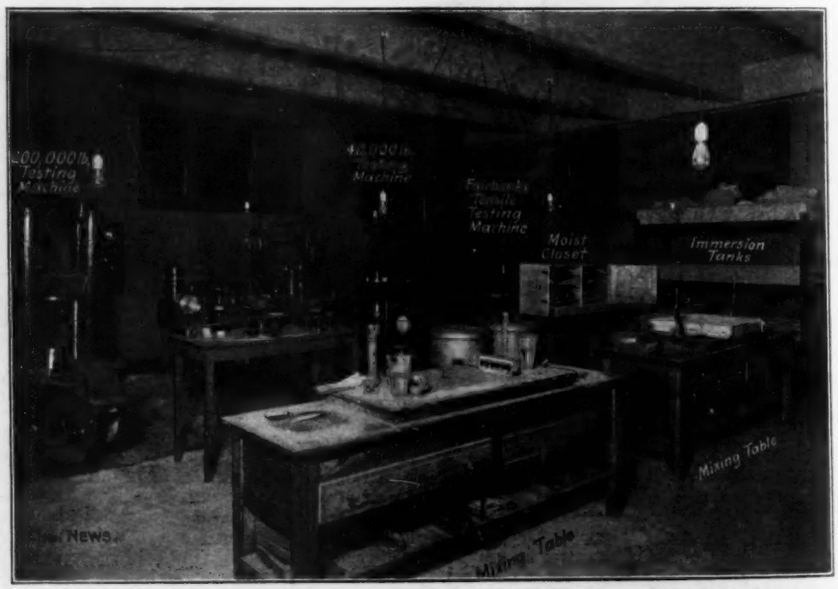


FIG. 3. RIGHT INTERIOR VIEW OF MODEL CEMENT-TESTING LABORATORY.

For accelerated tests, the apparatus shown by the drawings of Fig. 7 is provided. This is a steam test apparatus, provided with a constant level bottle for maintaining a uniform depth of water in the boiler. The test pats are placed on a wire rack set on brackets above the water level, and the boiler proper is made of sheet copper. The other details are clearly shown by the drawings.

The minor apparatus consists of two vicat needles for setting and consistency tests, of an Olsen needle and the other a needle imported from Germany; of a Bauschinger apparatus for measuring expansion; of standard sieves for cement and sand; of a Le Chatelier specific gravity apparatus, and of the usual briquette and brick molds, sample cans, trowels, gloves, etc., etc.

The work being done at the laboratory comprises a series of tests to determine the influence of the personal equation, and a series of tests to determine the relative value of natural sands from different sections of the country. These comparisons will be carried through the whole list of normal commercial tests and a number of special tests.

OUTSIDE EXHIBITS.—In the rear of the Cement Building are the outside exhibits, which serve to illustrate a few of the many uses to which Portland cement is put and some of the methods employed in mixing and molding.

Reinforced concrete construction is illustrated by a cantilever beam 33 ft. long, having a stairway suspended to and leading to the top. The beam is of T-section, the flanges forming a walk along the top of the beam. The beam has a dead load of 200 lbs. per sq. ft., and is designed for a uniform load of 100 lbs. The beam will be tested to destruction towards the close of the Exposition. The Hennebique Construction Co., of New York, is the exhibitor.

Adjoining this cantilever is an exhibit of the Siegwart hollow reinforced beams, exhibited by John E. Olsen, of New York. These beams are cast in a solid mass, the reinforcement and cores being placed in position; the concrete is then filled in. When sufficiently hardened the cores are withdrawn and the beams feathered apart. It is the intention to carry the beams in stock for various floor loads and spans, and when shipped to a building they are placed in position and grouted together. The beams in the exhibit were shipped from New York and so erected. The floor is about 2 ft. wide, has a clear span of 17 ft., and is carrying a uniform load of over 200 lbs. per sq. ft.

The Trussed Concrete Steel Co., of Detroit, Mich., has made a series of test beams, has erected a floor system, being a combination of hollow tile and concrete beams, and also some columns supporting a beam, all serving to illustrate the "Kahn System." The beams will be tested towards the close of the fair.

The Concrete Steel Engineering Co., of New York, exhibit the Thacher floor unit and Visintini beams.

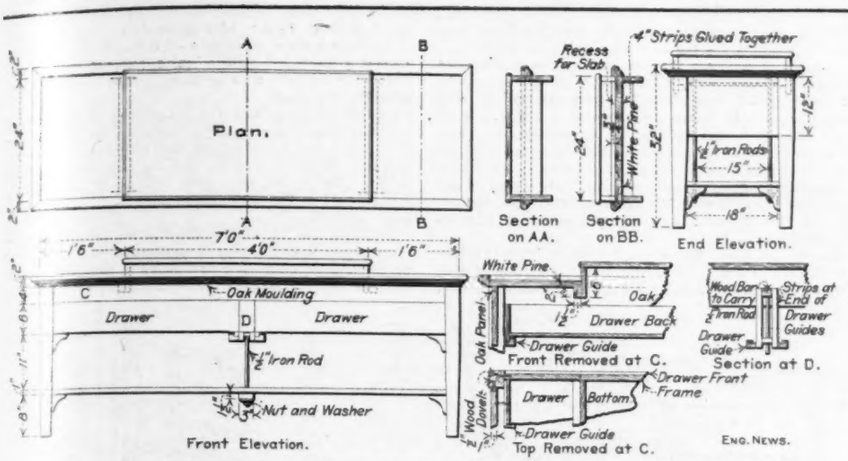


FIG. 4. DETAILS OF MIXING AND MOLDING TABLE FOR MODEL CEMENT-TESTING LABORATORY.

The Roebling Construction Co., of New York, has erected a floor panel between two steel beams, which rest on concrete piers, to show their system of fireproof flooring.

The Truss Metal Lath Co., of New York, exhibits a section of partition illustrating the use of the Truss metal lath.

Ornamental work in cement is illustrated by the Algonite Stone Co., of St. Louis. The exhibit comprises a porch consisting of two columns and two pilasters, supporting a pediment and roof; these pilasters and columns are connected on each side of the porch by a balustrade. Several steps lead to the porch floor. They also exhibit a Corinthian column cap, and a cap for a pilaster and other forms of artificial stone work.

The National Art Stone Co., of Chester, Pa., display an ornamental mantel and a column; the feature of the exhibit is the extremely low percentage of cement used. Samples are exhibited containing only 3, 7 and 10% of cement, and 97%, 93% and 90% of sand.

The Art Mosaic Tile Co., of St. Louis, exhibit mosaic work in Portland cement; they have a fireplace, a kitchen sink and flooring tiles.

The Vulcanite Paving Co., of Philadelphia, exhibit a section of granolithic pavement, and a section of steel-bound concrete curb.

Concrete railroad ties are exhibited by Casper Buhrer, of Sandusky, O., and Frank Ford, of Albion, Mich. The former is made of old rails with the head embedded in the concrete and with the flange up, the latter serving as the bearing and means of holding the rail in place. The Ford tie is in two pieces, connected by a rod, insulated at the connection. Reinforced concrete fence posts are exhibited by H. T. McCarthy, of Detroit, Mich., and a reinforced concrete burial vault is shown by Geo. A. Rackle & Sons, of Cleveland, O.

The Kielberg pipe, of Danish manufacture (made with a hydraulic press), is exhibited by F. L. Smidth & Co., of New York City, the American agents. A number of these pipe will be tested towards the close of the Exposition. The Wilson Baillie Co. of Brooklyn, N.Y., exhibit the standard egg-shaped sewer-pipe used by the City of Brooklyn.

Hollow building block machines are exhibited by H. S. Palmer, of Washington, D. C., the Cement Machinery Co., of Jackson, Mich., and the Burlington Block Machine Co., of Burlington, Ia. These machines are in operation daily, demonstrating the manufacture of cement blocks. A two-piece block is exhibited by the American Hydraulic Stone Co., of Denver, Colo. Walls of various thickness from 3 ins. and upwards are shown.

Cement brick and cement paving tile machines are exhibited by A. D. Mackay, of Chicago, Ill., while cement roofing tile is displayed by the American Cement Tile Mfg. Co., of Wampum, Pa., Brock Bros., of St. Louis, and Furman Construction Co., of Detroit, Mich. The Brock tile is plain, the American is made corrugated, diamond-shaped, and in the 4-ft. plain form, while the Furman is diamond-shaped.

The Municipal Engineering & Contracting Co., the McKelvey Concrete Mixer Co. of Chicago, have exhibits of concrete mixers, and the Contractors Plant Co. of Boston, Mass. The first named is a cubical mixer mounted on wheels, and is complete with boiler and engine for operating. The McKelvey mixer is a hand-driven barrel machine, while the Contractors Plant Co.'s is a portable gravity mixer.

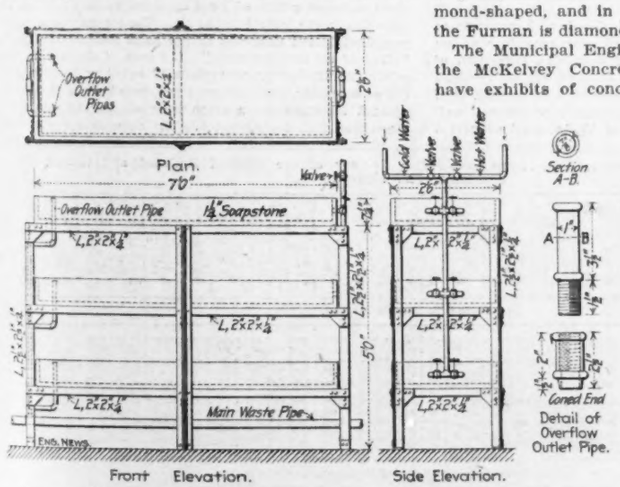


FIG. 6. DETAILS OF IMMERSION TANKS FOR MODEL CEMENT-TESTING LABORATORY.

THE REDUCTION OF RISK FROM FIRES CAUSED BY SPONTANEOUS COMBUSTION OF SOFT COAL.

The publication of an article on Spontaneous Combustion in Soft Coal, by Mr. Alfred O. Doane, in Engineering News of Aug. 18, has brought out some pertinent material issued by the Home Insurance Co., of New York, and prepared by Mr. F. M. Griswold, General Inspector for that company. The interest lies in the educational work in which this and other fire insurance companies are engaged in an effort to reduce the enormous annual fire loss in this country. These companies are convinced that much of this waste of capital is due directly to ignorance and carelessness on the part of the public—and that it is preventable.

The bulletin on bituminous coal-fires, sent out by the Home Insurance Co., is called forth by the frequency of fires in deposits of bituminous, or soft coal, generally justly ascribed to spontaneous combustion. This liability to spontaneous ignition and combustion appears to be most marked in free-burning, or so-called "high steaming" coals, including "gas coal." These coals usually contain a large percentage of volatile constituents in combination with a modicum of oxygen; and the tendency to ignition is particularly aggravated when they contain such impurities as lignites or sulphur in any form, but especially when iron-pyrites, or sulphuret of iron, is present. The dirty, or "mine-run" coal is the most dangerous, as the finer particles, due to disintegration, sift to the bottom of the pile and are there subject to the effect of pressure and consequent heating.

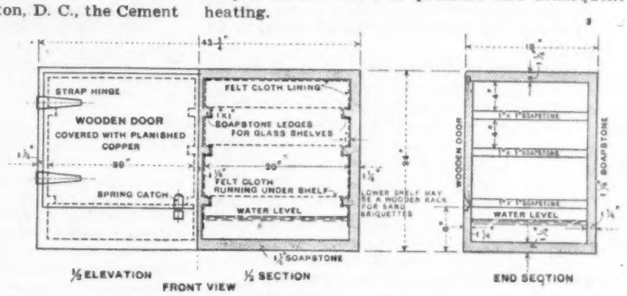


FIG. 5. DETAILS OF MOIST CLOSET FOR MODEL CEMENT-TESTING LABORATORY.

No entirely satisfactory explanation of the spontaneous ignition of bituminous coal has ever been formulated. But the best authorities agree in the conclusion that the heating can be ascribed to chemical changes in the substance, resulting from the absorption of oxygen; and that this absorptive quality of the carbon constituents increases with any rise in temperature in the mass; whether this rise is due to chemical action resulting from slow oxidation, or to mechanical force exerted through pressure, and these conditions are considerably stimulated in action by the presence of pyrites or moisture. Some authorities assume that anything in excess of 2½% of sulphur, in the pyrites, is indicative of danger in respect to heating.

Practical experience in coal fires due to spon-

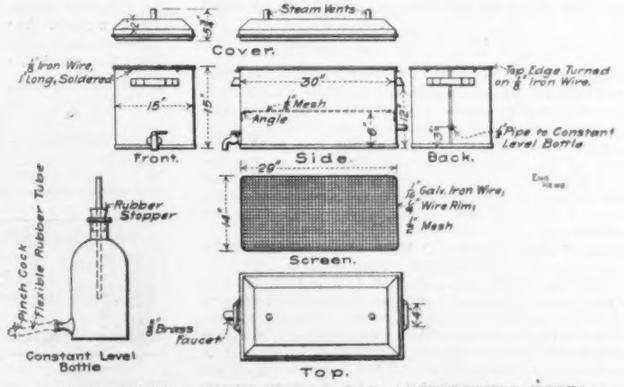


FIG. 7. DETAILS OF APPARATUS FOR ACCELERATED TESTS, MODEL CEMENT-TESTING LABORATORY.

taneous combustion indicates a varying propensity to "heat" in the several classes of bituminous coal used in the Eastern and Middle States. The order of susceptibility in these coals is about as follows: Nova Scotia, or Dominion; Pocahontas, W. Va.; New River, W. Va.; Clearfield, Pa.; Berwind-White, Pa.; Pittsburgh, Pa.; George's Creek, Md. The last-named coal is limited in volume, but no fire properly ascribable to spontaneous combustion has yet been reported for this grade of coal.

The bulletin then gives the following advice as gleaned from experience in handling and storing soft coal by steam railways, and remarks that the observance of these precautions tends to minimize if not entirely prevent the occurrence of self-ignition fires:

Accumulations of bituminous coal—in any one pile, heap, pocket or bunker, should not exceed 1,500 tons in volume. When more than one such pile is necessary, there should be a clear space of not less than 5 ft. between the piles; this space to be kept absolutely free for complete ventilation and the dispersion of occluded gases. No such pile, of 1,500 tons or less, should be more than 12 ft. higher when "trimmed-off." Where the pile is made by dumping from a trestle, the extreme height should not exceed 15 ft. at the apex of the triangle produced by this method of deposit.

Where coal is stowed under shelter or inside of a structure, most perfect ventilation should be secured, to facilitate the escape of gas.

Wet coal, especially when wetted by snow and ice, should be so disposed of as to be used first. If stowage is unavoidable, such wet coal should form the top of a pile and be spread as thinly as possible.

All bituminous coal piles, large or small, should be frequently and regularly "rod-tested," in order to discover any tendency toward dangerous heating. The danger line should be set at about 160° Fahr.; and where this degree of heat is reached piping should be inserted enclosing a self-registering thermometer. This precaution will enable the owner to take prompt and effective measures of relief without wasting time and the unnecessary handling of material to locate the trouble.

It might be here mentioned that in addition to the bulletin quoted above the Home Insurance Co. has issued other publications upon similar topics and with a like purpose. Noting these by title only, we have a careful handbook for special agents and inspectors relating to the Comparative Values of the Details in Buildings of Fire-Resisting Design. The others relate to the comparative fire hazard of glass-works, making other than plate or window-glass, lumber-drying plants, and fertilizer and sulphuric acid works. While these publications are primarily intended for the use of the company's inspectors, they contain much that is useful to the designer and operator of establishments of this character.

RECENT TESTS OF CENTRIFUGAL MINE VENTILATING FANS.*

By R. V. Norris,† M. Am. Inst. M. E.

In a former paper on this subject,‡ I summarized Murgue's theory and experiments, and presented a large number of tests of mine-fans, stating some conclusions, "more or less warranted by the evidence," as to design, capacity and efficiency. Since that time the very valuable report of Mr. M. Walton Brown,‖ Committee on Mechanical Ventilators, Institution of Mining Engineers, has appeared, and I have made additional experiments which tend to clear up some points in connection with this subject.

DESIGN AND TESTS OF THE LUKE-FIDLER COLLIERY FAN.

The Luke-Fidler fan (see Figs. 1, 2 and 3), built to replace an old wooden fan at a very fiery colliery, presents some peculiarities in design. Being constructed of concrete and iron, it is absolutely fireproof; and, with the addition of the large explosion doors over the airshaft, it is hoped also explosion proof. The construction of the fan permits the changing of the action in a few moments, from exhausting to blowing, by merely opening the double reversing doors in the air way and raising the movable sheet in the chimney (shown by the dotted lines in Figs. 1 and 2), both being done from the outside without necessarily stopping the fan or even altering its speed.

In order to facilitate the reversing and to improve the blowing passages, the fan, originally designed as shown in Figs. 4 and 5, was turned 30° from the vertical, thus giving an inclined chimney-flue, shown in Fig. 1. In all other respects it was designed for an exhaust fan which is its main work, the blowing feature being intended for use only in such emergencies, as mine fires or explosions. There being a material difference in the design of an exhaust fan and a blowing fan, no compromise was attempted, and high efficiency as a blower was neither expected nor attained.

The general dimensions of the fan are as follows: Diameter, 18 ft.; width, 7 ft.; double intake, diameter of each eye, 9 ft.; 8 blades, 7 ft. by 5.4 ft., of 3/16-in. steel, each stiffened by two 2.5-in. by 2.5-in. by 0.25-in. angles; flat arms, 3.5-in. by ⅝-in. (as indicated in Fig. 1), two 4-pointed spiders, 4.5 ft. centers, all connected with fitted bolts in reamed holes; spiral casing ¼-in. sheet-steel stiffened with 3-in. by 4-in. T-iron; casing concentric for 45° from the point of cut-off; balance of spiral calculated for 125,000 cu. ft. of air at 90 rev. per min., by the formula,

$$E = \frac{Q}{\pi DNB}.$$

E = expansion at cut-off. Q = quantity in cu. ft. per min. D = diameter of fan in feet. N = rev. per min. B = breadth of fan-blade in feet.

The figures obtained by this formula give a velocity of air at the cut-off equal to that of the tips of the blades; and it has been my practice to add 20% to the value of E thus found, in order to allow for the slowing of the air in the casing.

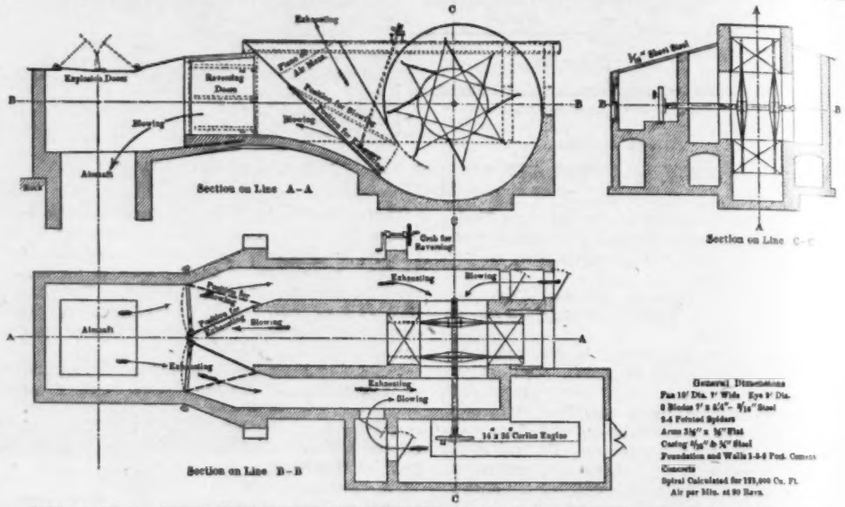
The airshaft, 10 ft. square, is lined with concrete to the rock, the explosion doors directly over it open an area 11 ft. 3 ins. by 12 ft.; the foundations, lower part of casing walls, and floors of the building are of Portland cement concrete, in the proportions of 1 of cement, 3 of sand, and 6 of broken stone thoroughly mixed and well rammed in place. The roof is of 3/16-in. steel supported on 3 by 4-in. T-iron, with cover-plates over [illegible] joints. The entrance doors are of sheet-steel 3/16-in. thick, [illegible] of the reversing doors are 0.25-in. thick stiffened with [illegible] iron. The engine is a 14 by 36-in. Corliss, direct-connected to the fan shaft.

SHAPE OF BLADES.—For the purpose of determining the influence of the shape of the blade tip on the efficiency of a fan, the tips were first bent backward [illegible] tangent, shown in Fig. 4, and the fan was carefully tested. The blades were then changed by straightening the tips forward to the radial shown in Fig. 5, and the tests were repeated.

METHOD OF TESTING.—The methods adopted for testing and the formulae used in calculation were those used by the English Committee as detailed in its report, except that, instead of changing the conditions in the [illegible], the airshaft was tightly closed and air drawn in through the explosion doors, the resistance being changed by varying the inlet-opening from the full opening of the explosion and entrance doors, down to an equivalent orifice of only 6 sq. ft.

This method of testing has many advantages over any test made directly on the mine, for the reason that it avoids all complications and uncertainties due to natural ventilation. Any desired condition can be established almost instantaneously, maintained as long as desired, and re-established if necessary to verify results. The temperature and barometric corrections are simpler. Any fan can be tested under identical conditions, without regard to its particular mine; and there is no interference with the regular ventilation of the mine.

The water-gage was taken close to the inlet by means of a pipe extending nearly to the outer end of the shaft, the end being protected by flannel so as to avoid variations due to the impact of the air. The air was necessarily measured in the chimney, at the point marked in Fig. 1, "plane of air measurement." The area of the flue at this point was divided by wires into 16 equal rectangles, and the anemometer, mounted on a rod, was held in the center of each rectangle for a given interval, usually one minute, timed with a stop-watch. The anemometer was then



FIGS. 1 TO 3. FIREPROOF REVERSIBLE VENTILATING FAN AT THE CAMERON AND LUKE-FIDLER COLLIERIES OF THE M. R. R. & M. CO., SHAMOKIN, PA.

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The water-gage was taken close to the inlet by means of a pipe extending nearly to the outer end of the shaft, the end being protected by flannel so as to avoid variations due to the impact of the air. The air was necessarily measured in the chimney, at the point marked in Fig. 1, "plane of air measurement." The area of the flue at this point was divided by wires into 16 equal rectangles, and the anemometer, mounted on a rod, was held in the center of each rectangle for a given interval, usually one minute, timed with a stop-watch. The anemometer was then

TABLE I.—Tests of 18-Ft. Fireproof Fan at the Luke-Fidler Colliery of the Mineral Railroad and Mining Co., Shamokin, Pa., 1903-04.

No. of Test.	Condition of Test.	Time.	Area of Office.	Corrected Anemom. Readings.	Area, Sq. Ft.	Cu. Ft. Air per Minute.	W. G. at Center.	W. G. at Inlet.	Thermometer. Dry.	Thermometer. Wet.	Bar'r.	B. P. M.	Rev. Air per Cu. Ft.	Peripheral Speed, per Minute.	Air per Min. at Normal Vol. lbs. & Cu. Ft.	W. G. at Norm. Vol. Inches.	Wt. of Air per Cu. Ft.	Equivalent Orifice, Sq. Ft.	H.P. in Air	I. H. P. of Engine Test.	Mech. Eff. Per Ct.	Indicator Cards. Nos.
	BLADE TIPS BENT BACK TO TANGENT.																					
1	Explosion doors open. Nov. 10.	2.45	125.04	2794	39.62	108,578	0.625	0.5	61	50	29.18	95	1960	4867	178,400	0.70	0.0743	30.65	16.41	46.06	34.9	8-12
2	" " " " "	2.57	125.04	2822	39.62	105,249	0.75	0.5	60	50	29.18	93	1912	4976	172,680	0.66	0.0743	31.56	16.57	71.40	23.2	13-17
3	" " " " "	4.12	62.06	2537	39.62	151,258	0.875	.....	59	48	29.18	81	1967	4560	168,030	1.09	0.0744	61.60	20.86	56.57	36.6	18-21
4	" " " " "	4.42	29.93	1917	39.62	114,292	2.000	.....	60	48	29.18	96	1191	5429	107,190	1.76	0.0746	30.97	26.02	66.25	54.3	26-29
5	" " " " "	5.00	15.62	1152	39.62	68,662	2.000	.....	62	45	29.18	91	726	5146	67,950	1.96	0.0756	18.75	21.65	43.80	49.4	30-34
6	" " " " "	5.09	6.81	692	39.62	41,257	2.125	.....	58	48	29.18	90	458	5090	41,257	2.12	0.0760	10.86	13.81	36.16	38.2	35-39
7	" " " " Nov. 11.	9.40	41.67	1711	39.62	102,910	1.063	.....	60	49	29.21	78	1598	4138	126,068	1.62	0.0775	38.64	17.8	39.8	44.7	44-45
8	" " " " "	9.45	41.67	2001	39.62	124,663	1.563	.....	60	49	29.21	90	1935	5000	124,663	1.57	0.0776	38.88	30.71	67.02	44.2	46-47
9	" " " " "	9.55	41.67	2227	39.62	132,774	1.938	.....	62	49	29.22	95	1289	5372	126,003	1.71	0.0774	39.25	40.5	77.7	52.2	48-49
10	" " " " "	10.05	61.04	3472	39.62	148,808	1.188	.....	63	41	29.22	90	1586	5060	143,808	1.19	0.0771	61.54	36.93	69.49	58.8	50-51
11	Explosion and fan doors open.	10.40	176.00	2908	39.62	173,375	0.5	.....	44	42	29.19	90	1926	5060	173,375	0.50	0.0767	95.26	18.66	78.37	17.4	55-57
	BLADE TIPS BENT FORWARD TO RADIAL.																					
12	Fan closed. Jan. 4.	2.00	.....	.....	.....	.....	2.2	2.1	12	.....	29.7	89	.....	4976	.....	2.31	0.0803	.....	.....	51.8	0	1-2
13	One explosion door open.	2.10	57.0	2045	39.75	182,569	1.4	1.2	13	.....	29.7	89.5	2044	5031	184,343	1.43	0.0834	62.39	40.3	101.7	39.6	3-4-5
14	" " " "	2.27	57.0	2004	39.75	177,018	1.84	1.2	14	.....	29.7	90.5	1908	5110	174,672	1.53	0.0832	57.63	43.0	99.1	43.4	6-7-8
15	" " " " 1/16" "	2.40	36.6	1462	39.75	88,307	2.3	2.3	14	.....	29.7	90.5	977	5107	88,081	2.29	0.0832	29.50	31.9	69.5	45.7	11-12
16	" " " " 1/8" "	3.02	12.0	963	39.75	58,101	2.51	2.4	16	.....	29.7	91.5	632	5488	57,115	2.32	0.0828	15.18	23.0	48.3	47.6	13-14
17	" " " " 3/16" "	3.13	6.0	533	39.75	31,675	2.64	2.5	16	.....	29.7	92	344	5203	31,040	2.46	0.0826	7.96	12.8	30.3	42.6	15-16
18	" " " " 1/4" "	3.25	30.6	2574	39.75	144,096	2.56	2.4	16	.....	29.7	90	1561	5429	139,945	2.38	0.0826	36.50	58.1	94.5	61.5	17-18
19	" " " " 3/4" "	3.35	30.6	2504	39.75	142,628	2.56	2.3	14	.....	29.7	90	1486	5429	139,365	2.36	0.0822	36.02	57.5	95.3	60.3	19-20
20	Explosion doors full open.	3.56	113.0	3274	39.75	197,551	0.92	0.7	13	.....	29.7	88	2245	4976	202,454	0.87	0.0834	85.29	58.6	94.5	30.3	23-24
21	" " " "	4.13	113.0	3308	39.75	157,174	0.57	0.4	12	.....	29.7	71½	2179	4050	197,435	0.59	0.0835	84.59	54.1	85.1	25.6	25-26
22	" " " "	4.26	170.0	1617	39.75	97,580	0.25	0.3	12	.....	29.7	48	2225	2691	191,292	0.56	0.0835	79.60	8.8	17.0	22.6	27-28-29
23	" " " closed. Jan. 5.	10.50	Mins.	826	39.75	50,411	2.15	.....	19	.....	29.7	89	567	5033	51,982	2.21	0.0800	14.44	17.1	37.5	45.6	30-31-32-33

* Normal velocity—4,100 ft. per min.

*A paper read at the Lake Superior meeting of the American Institute of Mining Engineers.

†Chief Engineer Penna. Coal Co., First National Bank Building, Wilkesbarre, Pa.

‡Trans., xx., 637.

‖Transactions of the Institution of Mining Engineers (London), vol. xvii., p. 482, and vol. xviii., p. 488.

moved to the center of the next square, and so on until all had been covered, the total reading for the 16 squares being used in determining the average velocity of the air-current.

All the measurements were checked by the use of two anemometers, either simultaneously, beginning at opposite sides, or by repeating the measurements. Both anemometers were tested and the proper corrections made to the readings obtained. The details and results of these tests are given in Table I., and the corresponding diagrams showing the curves of air delivered, the mechanical efficiencies, and the water gages for varying orifices, with blade-tips tangential and radial, are given in Figs. 7 and 8. From the data in Table I., it is apparent that the radial tip blades have a very decided advantage in quantity of air delivered, in the water gage maintained and in efficiency. These results agree with Mr. Murgue's theoretical investigation. The tangent tip, however, is a sufficiently common practice in the anthracite region to warrant the experimental investigation.

The diagrams show clearly the proper capacity of this particular fan, and it is interesting to note that the point of highest efficiency occurs about at the point where the curve of air delivered (called the characteristic curve of the fan by the Committee on Mechanical Ventilators) begins to spring downwards from a straight line.

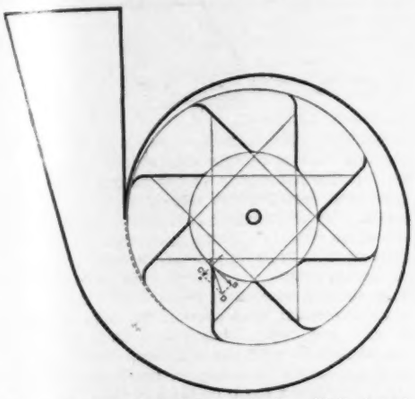


Fig. 4. Modified Guibal Spiral Casing, 18-ft. American Fan; Blade-tips Bent Backward to Tangent.

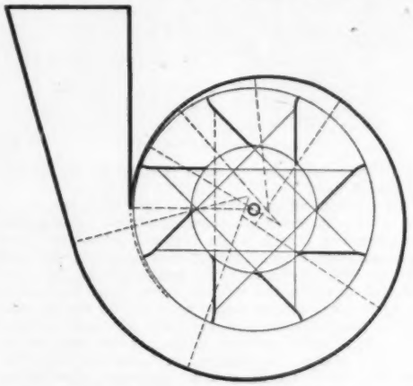


Fig. 5. Modified Guibal Spiral Casing, 18-ft. American Fan; Blade-tips Bent Forward to Radial.

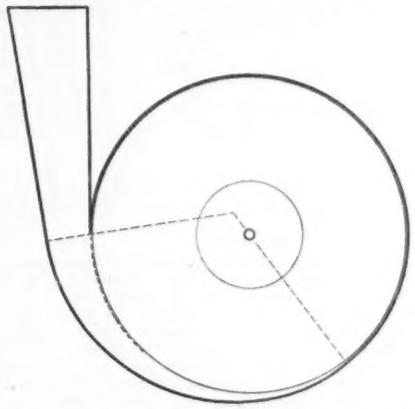


Fig. 6. Guibal Spiral Casing of a 36-ft. English Fan.

COMPARISON OF RESULTS.

In order to ascertain whether this result was accidental, or was characteristic of Guibal fans, I have plotted the curves of other fans in the diagram, Fig. 9, using the tests of the Committee on Mechanical Ventilators as well as a few from my former paper, adding also the curves of the Schelle, Waddle and Capell fans from the same sources. This diagram shows that each size and type of fan has its own characteristic curve, but that all fans of the same type, on mines of the same resistance, give practically the same quantity of air up to the capacity of each fan, when the peripheral speed of tips of blades is the same. Up to 45 sq. ft. equivalent orifice the curves of all the Guibal fans practically coincide; at this point the curve of the 18-ft. fan begins to deflect downwards, and, as the equivalent orifice is enlarged, the curves of the larger fans likewise drop.

The curves of the other three types of fans follow the same law, but unfortunately the available tests on varying sizes are too few to extend the curves in detail.

By comparing the efficiencies with the characteristic curves, three results are noteworthy, namely—1. Each size of fan reaches its best mechanical efficiency at about the point where its curve begins to vary from a straight line. 2. Each size of fan is at its best on mines of a certain equivalent orifice which for the highest efficiency can vary only within rather narrow limits. 3. The mechanical efficiency is poor, with fans too large for their mines as well as for those too small. These results show conclusively the absolute necessity of designing a fan with reference to its proposed work, and the futility of attempting to use a standard size or design of fan for different mines.

The curves of water gage (see Fig. 10), also, show that the depression of the same fan varies with different resistances, and illustrates the usefulness of the old term "manometric efficiency" which, in the case of the Fidler fan, varies from 73% at 8 sq. ft. to only 30% at 83 sq. ft. equivalent orifice. It is noticeable, however, that the fans with large spirals maintain a higher depression up to their capacity than do the English fans, and that the depression curves flatten and hold up better as the diameter of the fan increases.

It is also noticeable that the curves of each size of American fans lie above those of similar sizes of English ones. This result is probably due to two causes, 1, the shape of the casing, the English being constructed on Guibal's original lines, Fig. 6, with a close casing for about two-thirds of their circumference and a contracted outlet, and the American fans, Figs. 4 and 5, with spirals starting near the cut-off and expanding all around to the chimney; and, 2, the great variation in size of eye as compared with the diameter of the fan, the English fans having eyes rather less than one-third the diameter, while those of the American fans are at least one-half.

It is probably unfair to the Capell fan to include these old tests in the diagrams, especially as it is essentially a high-speed fan, and most of the tests available had to be reduced from periphery speeds of 7,000 ft. and upwards per min., but, as a request to the American agent of this fan for further data has met with no response, the old figures are given for what they are worth. It is interesting, however, to note that its characteristic curve, when reduced to the same speed, falls between those of the Guibal, and those of the Schiele and Waddle types.

The only available test of the new Sirocco fan is one made on a mine having 56 sq. ft. equivalent orifice, which shows that this 6.25-ft. fan gave about the same quantity of air as the larger Guibal fans; unfortunately, no figures of efficiency of the Sirocco fan are available.

In view of the further data now given, the unsatisfactory conclusions of my former paper should be revised; and the following are presented in their place.

1. INFLUENCE OF RESISTANCE OF MINE (EQUIVALENT ORIFICE) ON FAN.—The quantity of air delivered by any fan at a given speed is practically proportional to the equivalent orifice of the mine, up to the proper capacity of the fan; beyond this capacity the proportion rapidly decreases, as shown by the characteristic curves, tests 1 and 2.

The highest mechanical efficiency of any fan is reached with a mine which permits it to work at or near its proper capacity.

2. INFLUENCE OF THE DIAMETER OF A FAN ON ITS PERFORMANCE.—The proper capacity of a fan is largely dependent upon its diameter, hence the size should be determined with reference to the resistance of the mine upon which it is to work; the efficiency of any fan depending largely upon its working at or near its proper capacity.

3. INFLUENCE OF THE WIDTH OF A FAN UPON ITS PERFORMANCE.—Very little experimental data is available on this point, but it seems reasonable to believe that the cylindrical inner surface of the blades should be approximately equal to the clear area of the inlet eye or eyes. The fan, 20 ft. in diameter and 12 ft. wide, referred to in my earlier paper, was tried and proved a comparative failure; both the quantity of air delivered and the efficiency were low, due probably to its excessive width in comparison with its area of intake.

4. INFLUENCE OF SHAPE OF BLADES OF A FAN ON ITS PERFORMANCE.—The conclusive tests of the Fidler fan, Fig. 8, prove that the tips of the blades of a Guibal fan should be at least radial. No experiments are available, however, on blades bent forward beyond this point, though it seems reasonable to believe that the radial tips are correct. The gain in the quantity of air delivered, in mechanical efficiency, and in water gage, accomplished by changing the blade tips from tangential to radial position, is too great to have been the result of any possible error in experiment, especially as the tests were made by the same observers and apparatus upon the

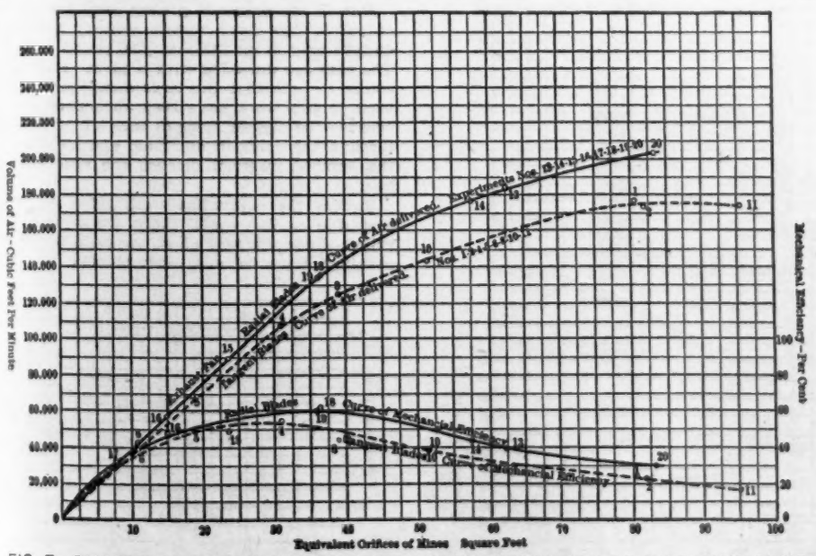


FIG. 7. DIAGRAM OF AIR DELIVERY AND MECHANICAL EFFICIENCY OF MINE FANS PLOTTED FROM RESULTS OF TESTS GIVEN IN TABLE I.

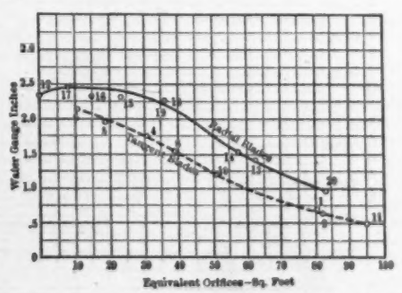


Fig. 8. Air Pressure by Water Gage of Mine Fans, Plotted from Table I.

TABLE II.—Comparative Tests of Mine Fans on Mines of Varying Resistance.

Reference Number.	Description of Fan.	Air-density Barom. Feet.	Area of Measurement. Sq. Ft.	Cu. Ft. Air per Minute.	Water Gauge. Orifice Inches.	Water Gauge. Drift Inches.	Thermometer. Dry.	Thermometer. Wet.	Bar. Inches.	Rev's per Minute. Fan.	Air per Revolution. Cu. Ft.	Periphery Speed. Feet per Min.	Air per Min. of 6,100 Ft. Periphery Speed. Cu. Ft.	W. G. of 5,100 Ft. Sphery Speed. Inches.	Avg. Weight of Air per Cu. Ft. Lbs.	Equivalent Orifice of Mine. Sq. Ft.	H.P. in Air-Test.	H.P. of Engine-Test.	Mechanical Efficiency. Per Cent.	
1	Great Western Colliery:	281	102.37	2,466	0.06	0.03	85½	61½	29.186	6½	256	848	14,096	1.807	0.0787	4.686	0.019	2.86	0.79	
2	Guibal fan, 40 ft. dia., 12 ft. wide;	3765	"	39,517	2.59	0.02	56¾	64	29.185	47½	805	5,016	32,643	1.800	0.0734	9.091	15.704	98.55	15.93	
3	dia. single intake, 14 ft.	5265	"	52,898	2.57	0.02	67	64½	29.185	68	1,125	6,032	45,585	1.826	0.0735	12.776	21.856	102.84	21.26	
4	Tested June 3, 1899.	2,055	"	210,388	2.44	1.88	64½	63½	29.18	40½	4,813	5,850	188,113	1.548	0.0735	51.301	80.86	146.12	55.32	
5		3,200	"	338,819	1.55	0.42	62½	60¾	29.20	41½	8,189	5,190	322,900	1.490	0.0738	103.988	82.49	161.72	51.13	
6	Felton Colliery:	3045	75.52	29,287	2.96	2.82	56	56	29.71	14½	522	5,121	29,536	1.969	0.0770	4.05	12.72	68.47	18.47	
7	Guibal fan, 36 ft. dia., 12 ft. wide;	3465	"	47,917	2.85	2.80	56½	56½	29.72	58½	806	6,023	36,342	2.045	0.0770	9.80	19.27	98.49	28.09	
8	dia. single intake, 13 ft.	3,177	"	170,966	2.65	2.60	61	61	29.82	61½	2,791	6,027	127,887	1.978	0.0767	34.81	38.34	174.95	58.21	
9	Test, January 19 and 20, 1899.	2,243	"	179,104	2.57	2.52	58½	58½	29.82	51½	3,426	5,810	164,568	1.930	0.0768	42.75	71.35	122.48	57.76	
10		3,241	"	254,516	1.96	1.85	65¼	62½	29.81	61½	4,906	5,868	221,320	1.494	0.0779	70.79	79.34	184.36	43.05	
11	Wingate Grange Colliery:	2941	89.40	25,159	1.69	1.59	91	79	29.46	46½	521	4,580	28,930	1.977	0.0745	7.66	6.32	38.30	22.49	
12	Guibal fan, 36 ft. dia., 12 ft. wide;	7091	"	62,543	1.34	1.25	92	91½	29.45	68	1,578	4,431	71,172	1.989	0.0743	19.28	15.14	78.45	52.88	
13	dia. single intake, 13 ft.	1,658	"	146,611	2.64	2.34	80½	79½	29.40	52½	2,806	5,909	126,517	1.966	0.0761	34.68	41.01	136.46	55.28	
14	Test, February 2 and 3, 1899.	1,847	"	186,756	1.61	1.44	79½	79	29.55	41½	3,815	4,665	149,494	1.928	0.0764	41.47	34.79	67.57	51.48	
15		1,036½	"	96,886	1.43	0.54	80½	79¾	29.20	28½	2,810	2,926	162,760	1.852	0.0742	45.91	8.98	17.46	50.83	
16		2,041½	"	180,469	1.13	0.99	79½	79	29.66	37½	4,764	4,286	214,859	1.586	0.0758	65.50	31.94	69.08	46.25	
17	Guibal fan, 25' dia., 8' wide, 11' 6" intake.	.....	"	127,760	2.55	.....	.....	.....	.....	70	1,825	5,498	127,860	2.19	.....	32.0	50.35	67.44	82.07	No. 3 B
18	" " 25' " 8' " 11' 6" "	.....	.....	225,288	1.42	.....	.....	.....	.....	92	3,634	5,867	226,160	1.59	.....	73.0	51.44	82.80	61.74	" 4
19	" " 25' " 10' " 12' 0" "	.....	.....	196,288	1.80	.....	.....	.....	.....	73	2,880	5,504	179,250	2.14	.....	44.0	75.69	101.07	75.69	" 10 B
20	" " 25' " 8' " 12' 0" "	.....	.....	202,280	1.80	.....	.....	.....	.....	70	2,904	5,495	188,670	1.55	.....	52.0	57.70	107.30	54.00	" 13 I
21	" " 25' " 11' 6"* 17' 6" "	.....	.....	228,850	1.95	.....	.....	.....	.....	66	4,975	5,060	230,600	1.59	.....	64.0	70.02	98.45	71.75	" 8
22	" " 25' " 11' 6"* 17' 6" "	.....	.....	224,850	1.85	.....	.....	.....	.....	65	4,648	5,060	236,690	1.86	.....	68.0	69.46	94.60	72.37	" 9 A
23	" " 25' " 11' 6"* 17' 6" "	.....	.....	257,525	2.09	.....	.....	.....	.....	50	5,150	5,060	238,780	2.05	.....	68.5	97.49	151.30	74.28	" 11½ B
24	" " 25' " 12' 0"* 17' 6" "	.....	.....	177,000	2.25	.....	.....	.....	.....	49	3,887	5,278	171,890	2.10	.....	67.0	62.76	82.30	76.30	" 11 A

same fan changed only in this particular, and free from any influence of the mine.

The inner edges of the blades should have such an inclination as to receive the air without shock. This effect seems to be attained by making the angle between the inside of blades and a tangent to the inner surface the resultant of the radial speed of the air at this point, and the tangential speed of the inner edges of the blades.

5. INFLUENCE OF THE SHAPE OF THE SPIRAL CASING OF A FAN ON ITS PERFORMANCE.—This influence is considerable, the large casing, Figs. 4 and 5, not only giving higher efficiencies than the close casing, Fig. 6, but higher water gage and larger volumes of air under the same conditions; the American Guibal 25-ft. fan being nearly equal, in the quantity of air, to the English 36-ft., and the American 35-ft. fan to the English 40-ft. The American fans are also superior in the water gage maintained.

The spiral casing for any fan should be designed with reference to its expected work.

6. INFLUENCE OF THE DIAMETER OF INLET EYE OF A FAN ON ITS PERFORMANCE.—This influence may be considerable, and with the casing it may be taken as a reason for the superiority of the American type. Experimental data on this subject are woefully lacking, but it seems probable that the ratio of the diameter of inlet to the diameter of fan should vary with the proposed work. Fans with large inlets may probably prove suitable for large quantities of air and low water gages, and those with relatively small inlets for the reverse conditions. Relatively large inlets would certainly decrease the interval resistance of a fan and thereby increase its proper capacity.

7. INFLUENCE OF THE SPEED OF A FAN UPON ITS PERFORMANCE.—An analysis of many experiments shows, that while the air delivered by any fan is practically directly proportional to its speed and that the water gage given with the same equivalent orifice is proportional to the square of the speed, the mechanical efficiency is much affected by the speed at which a fan is run. Tests Nos. 20, 21 and 22 of the Fidler fan, and No. 13A to 13J of the South West Coal & Coke Co. fan, show clearly the variation in efficiency due to speed, and also verify experimentally the laws relating to air delivery and water gage.

The question of the proper speed for each type and size of fan requires much additional experiment for its proper determination, though it still is probable that for Guibal fans, at least, a peripheral speed of from 5,000 to 6,000 ft. per min. is the best, the proper size of fan being adopted to obtain the required air at about that speed, and more than one fan used if necessary for this purpose.

It is hoped that this present paper may awaken interest in this much-neglected subject, as only by the co-operation of many engineers can sufficient data be accumulated to settle the many vexed questions pertaining to the design of a ventilating fan. This desired result may best be accomplished by the publication of the characteristic efficiency and water gage curves of various sizes and designs of fans, so that, from the whole, it may be possible, eventually, to select a fan best suited to the particular conditions under which it is required to operate.

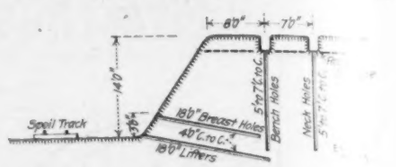
PROGRESS OF THE EXCAVATION WORK AT THE N. Y. CENTRAL & HUDSON RIVER R. R. TERMINAL.

Excavation work has been in progress for 13 months on the N. Y. C. & H. R. R. R. terminal in New York City. The contract involves the excavation of about one and a half million cubic yards of rock and earth, of which only about a quarter of a million yards have thus far been taken out. The great bulk of the material has been loaded into cars with steam shovels, of which there are three, two Marion and one Bucyrus. One shovel works two shifts daily, but the other two shovels work only day shifts. They are idle fully half of the working time waiting for cars, the enormous traffic of the two railways entering the New York station being such as to cause many delays in the moving of work trains. To facilitate night work, the shovels are being equipped with electric arc lights, suspended from light gallows frames mounted at the forward end of the shovel. The wire cable, from which the electricity is drawn, is carried by a reel mounted on the gallows frame, and is fed out as the shovel advances.

The excavation is, for the most part, mica-schist and gneiss, which is overlaid by a few feet of earth. A typical cross-section showing the arrangement of drill holes may be seen in the accompanying illustration. In some places the working face is considerably higher than shown. The spoil track shown in the sketch is standard gage, and parallels the side of a cut for its full length. By the method of blasting employed, little or no rock is thrown upon this track. Two rows of drill holes (a row of "lifters" and a row of "breast holes") are drilled about 18 ft. deep, the holes in each row being about 4 ft. c. to c. Two rows of vertical holes about 7 ft. apart (the holes in each row being from 5 to 7 ft. c. to c.) are drilled parallel with the side face. This side face, as shown in the sketch, does not break out vertically, but has a slope of from ½ to 1, to ¾ to 1. The bench and neck holes are "sprung" or "squibbed out" at the bottom by small blasts, so as to make chambers in which large charges of explosives may be placed. All the holes are charged with Joveite, of a grade equivalent to 60 per cent. dynamite, and are fired in small groups for a distance of several hundred feet in advance of the shovel. The explosion lifts the whole mass several feet into the air without hurling the fragments, and effectively shatters the rock. This mica-schist is an exceedingly tough rock, having a tendency to break out in long slabs, for which reason steam shovels with very large buckets are advisable. Two of the shovels are 70-ton, provided with 2½-yd. buckets; the third, a Marion shovel, model 91, weighs 95 tons and has a 5½-yd. bucket. To show what a shovel of this capacity can do on a short unobstructed run, even with very large slabs of rock, it is noteworthy that a flat car (with sideboards) was loaded in 12 minutes. Another car was loaded in 20 minutes, but there were many delays due to chaining out very large stones, to the breaking of sideboards on the car, etc. One of the small shovels working in rock broken up smaller, loaded two cars in 40 minutes. Each of these cars holds about 13 cu. yds. of solid rock on the average.

Due to the delays in getting cars, the average output of the three shovels working two shifts daily is only about 1,000 cu. yds. per 24-hour day. The cars are unloaded with a plow. It is interesting to note that the large dump cars at first used on this work have been abandoned, largely because the biggest slabs of rock would not pass through the doors in dumping, and also because the dumped material blocked the rails more or less.

Besides the excavation work going on in the east side of the yards, there are several large gangs of men engaged along the main tracks, excavating trenches and underpinning buildings



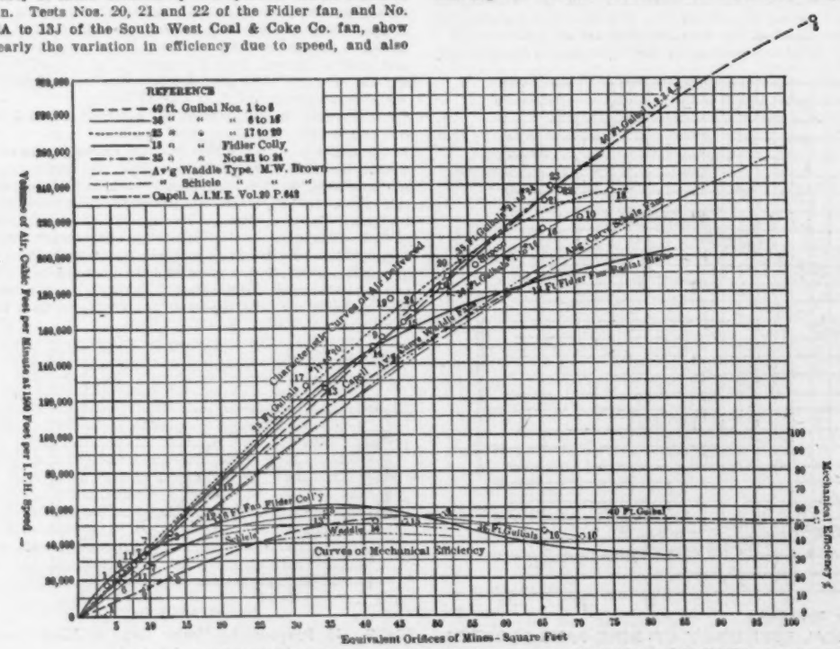

REFERENCE
40 ft. Guibal Nos. 1 to 8
36 " " " 6 to 16
25 " " " 17 to 20
13 " " Fidler Colly.
35 " " Nos. 21 to 24
Av'g Waddle Type. M.W. Brown
" Schiele " " "
Capell. A.I.M.E. Vol. 20 P. 648
Curves of Mechanical Efficiency
Equivalent Orifices of Mines - Square Feet
Mechanical Efficiency %

FIG. 9. CURVES OF TESTS OF MINE FANS FROM VARIOUS SOURCES.

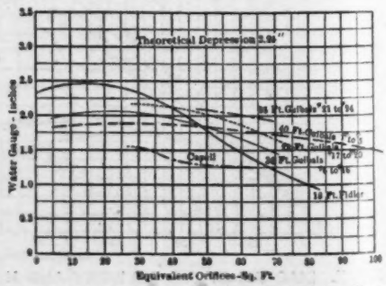


Fig. 10. Air Pressure Curves of Various Fans.

The excavated material is loaded by hand into skips, and hoisted with derricks and locomotive cranes.

The O'Rourke Engineering Construction Co., of New York, have the contract for this terminal work; Mr. A. B. Corthell is Terminal Engineer, and Mr. F. C. Shepherd is First Assistant Engineer.

THE NEW WATER TANK AND SUPPORTING TOWER FOR EAST PROVIDENCE, R. I.*

By F. M. Bowman,† M. Am. Soc. M. E.

During the autumn of this year (1904) the East Providence Water Co. completed its new water tank and tower at East Providence, R. I., and as the structure is in several respects the largest of its type, a description of it in detail will no doubt be of interest.

The tower is 135 ft. high from base of column to base of tank, and the tank is 50 ft. in diameter by 70 ft. high, with a capacity of 1,000,000 gallons. The height from the base is therefore 205 ft. for water pressure, and the total height to the top of the finial is 234 ft. The structure makes a water reservoir and fire protection for East Providence and Riverside, R. I., it being located on an elevated point about half way between the two places; the pumping station, however, being located some three miles from the site.

For ten years previous to the completion of this structure these cities were supplied with water direct from the pumps, there being no reservoir of any kind.* The pressure now obtained is great and the residents find it necessary to look carefully to their plumbing.

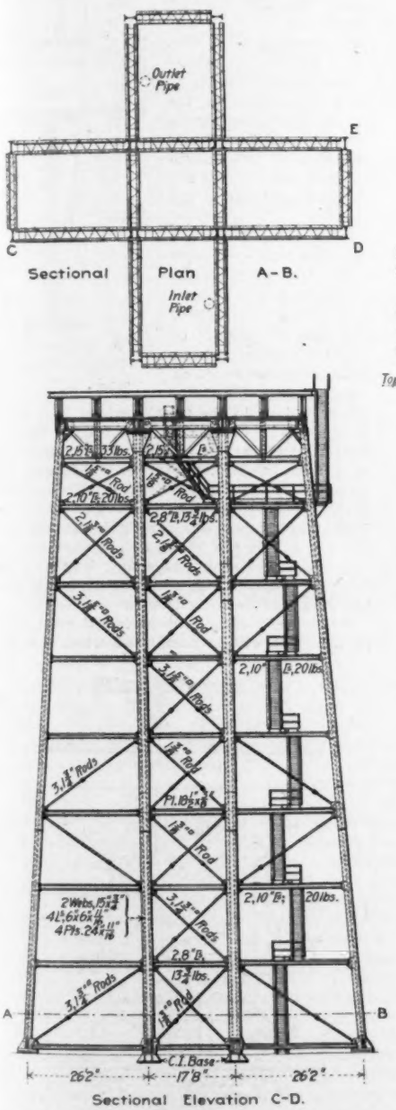


FIG. 1. PLAN AND ELEVATIONS OF WATER TOWER FOR THE EAST PROVIDENCE WATER CO., EAST PROVIDENCE, R. I.

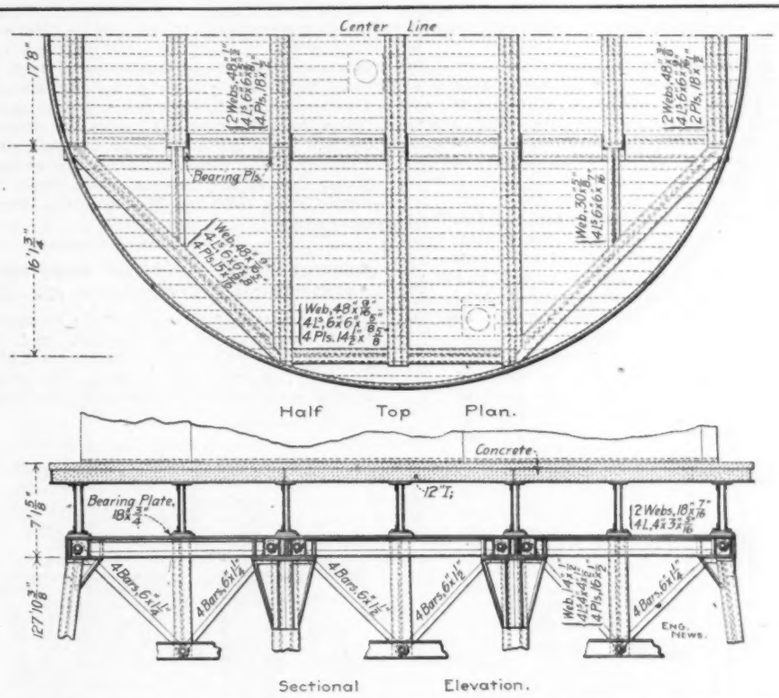


FIG. 2. PLAN AND SECTION OF CONCRETE TANK PLATFORM, EAST PROVIDENCE WATER TOWER.

Referring to the details of design (Figs. 1, 3, 5 and 8), it is to be noted that the tower has four center and eight outside columns. Most of the direct load is carried on the four center columns, each column carrying, approximately, 700 tons. The location is such that the foundations are carried to solid rock, making ideal construction. The foundations are of concrete, except that a granite cap stone in one piece is used under each of the twelve columns, this cap stone in the case of each center column being 3 × 6½ × 6½ ft., weighing 11 tons. The lower section of this column weighs 12 tons, exclusive of the cast-iron base (see Fig. 4 for base), and consists of two 15 × ¾-in. web plates, four 6 × 6 × 11-16-in. flange angles and four 24 × 11-16-in. cover plates.

The wind pressure was figured at 50 lbs. per sq. ft. of exposed surface for the tower, and in the case of the tank the pressure was assumed as two-thirds of this load on the diametrical area instead of the usual one-half. The horizontal wind strains from the tank are carried almost entirely by the eight outside columns, the columns acting as the top and bottom chord of the cantilever girder with panel points at every third horizontal strut, the cantilever being fixed to the ground by the anchor bolts.

The floorbeams for the platform (Fig. 2) are 12-in. I-beams, carried on box girders spaced about 9 ft. c. to c., these in turn being carried on pin-connected girders; the load from the box girders being transmitted by direct bearing to the center post of the pin-connected girders. The platform itself is made of concrete 16 ins. thick, this making the bottom of the tank frost proof and protecting the lower surface of the same from rust. The concrete is carried to the hand railing along the edge of the platform and forms a foot-walk around the tank.

The bottom of the tank is ½-in. steel plate; the sides vary in thickness from 15-16-in. for the bottom ring to ⅜-in. for the top ring. The horizontal seams are lap joints with two rows of rivets ⅞-in. in diameter for the lower rings. The upper seams have one row of rivets only, with ¾-in. rivets. The vertical seams are butt joints for the lower eight courses and lap for the upper six. The butt joints have two splice plates, the inner one about twice as wide as the outer to increase the percentage efficiency of the joint. The diameters of rivets vary from 1 in. for the lower ring to ¾-in. at the top. The tank is anchored to the tower with eight bolts 2 ins. in diameter, connected to the lower course of side plates and

*A paper read before the New England Water-Works Association, Nov. 9, 1904.

†Structural Engineer, Ritter-Conley Manufacturing Co., 56 Water St., Pittsburg, Pa.

(*A stand-pipe 40 ft. in diameter and 125 ft. high was formerly in use here. On Sept. 7, 1893, the upper portion of the stand-pipe was blown in, and on Jan. 19, 1894, the stand-pipe collapsed. See Eng. News, Sept. 14 and 21, 1893, and Jan. 25, Feb. 8 and 22, March 1 and 8, 1894. —Ed.)

to the web plates of the supporting girders (Fig. 6).

A balcony is provided around the side at the top of the tank. An opening 30 ins. deep is allowed between the top of the shell and the roof line for ventilation, the opening being covered with a heavy wire screen. The roof (see Fig. 8) consists of ⅛-in. plates, riveted together and supported on steel trusses, the finial being a 24-in. copper ball.

The inlet and outlet pipes are 14¼ ins. inside diameter steel pipe, supported on a suitable foundation in a pit where the gate valve is located; they are attached to the tower by steel ties at the cross struts and a slip expansion joint is provided at the top section. Steel flanges are used for

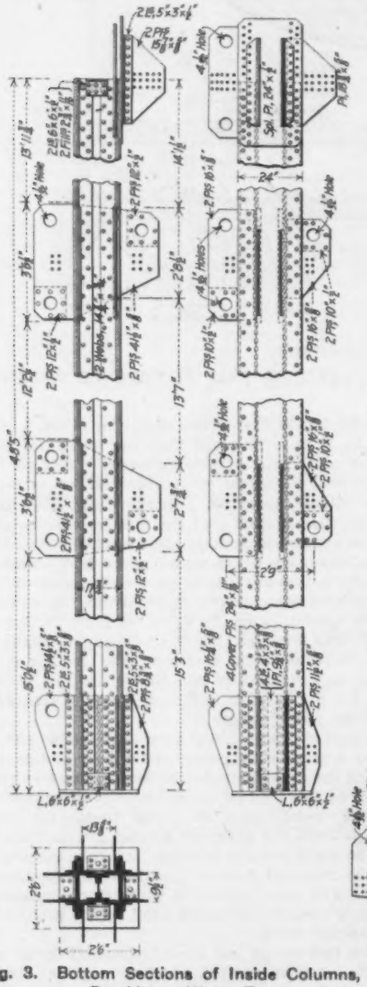


Fig. 3. Bottom Sections of Inside Columns, East Providence Water Tower.

pipe connections, and the pipe is covered with 3 ins. of asbestos, the asbestos being in turn protected by a sheet metal cylinder ⅛-in. thick, made in two halves, with flanges projecting outward for bolting the sections together.

Stairs 30 ins. wide are furnished to the base of the tank; the tank is provided with a vertical ladder 15 ins. wide, with a balcony half way up the tank as a rest. A separate stair is also provided to the manhole in the bottom of the tank, this manhole, 16 ins. in diameter, being for ingress and egress to the inside of the tank for cleaning and painting.

The material of the tower is medium and that of the tank soft open-hearth steel. The structure is designed for a factor of safety of four, the tension unit strain in the tower being 14,000 lbs. and in the tank 11,000 lbs. per sq. in. The pressure on the column bases on the granite caps does not exceed 400 lbs., and that of the granite on the concrete 250 lbs. per sq. in.

The erection had to be carried on carefully, due to the great height and prevalent high winds, and the work was slow and expensive for these reasons and because it was impossible to carry on the raising of material and the driving of rivets at the same time. The raising of material had to be abandoned when the riveting was in progress, as otherwise the riveters were liable to accident and injury. The work was carried on under the supervision of the engineers of the East Providence Water Co., Messrs. Bushnell and Wood. The Riter-Conley Mfg. Co., Pittsburg, Pa., were the contractors.

PROGRESS OF THE EXCAVATION WORK FOR THE PENNSYLVANIA R. R. TERMINAL IN N. Y. CITY.

The excavation for the site of the Pennsylvania R. R. station, between Seventh and Ninth Avenues, from Thirty-first to Thirty-third Streets, is now in full swing. About one and three-quarter million cubic yards of earth and rock must be

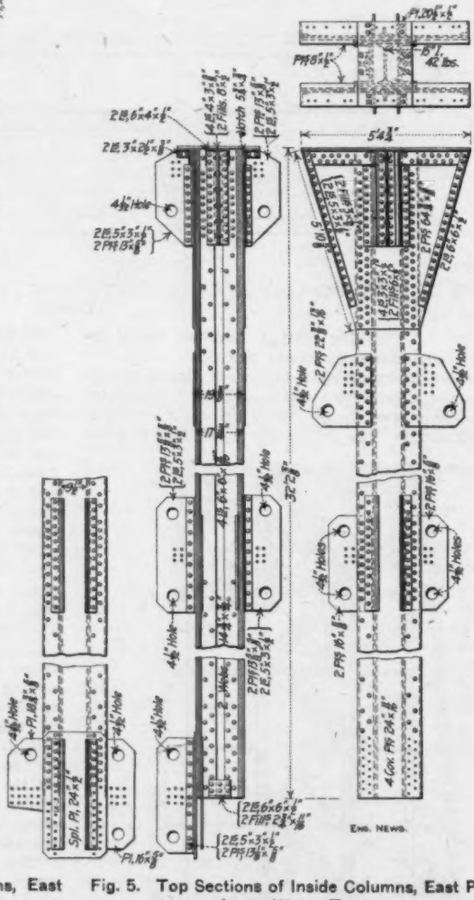


Fig. 5. Top Sections of Inside Columns, East Providence Water Tower.

removed, and approximately 10 per cent. of this yardage has already been taken out, the greater portion of the excavation, thus far, being earth. Three steam shovels are engaged in loading dump wagons, for, up to the present, it has been impossible to install tracks for conveying the material to the dock; but within the next six or eight weeks it is hoped to have a dock prepared and track laid to it. This track will pass underneath Ninth Avenue, and down Thirty-second Street, crossing the other avenues on a trestle that will be built to the water front.*

The site for the compressed air plant is being rapidly excavated in rock at Ninth Avenue. At present the drills are operated by steam. In this connection it is interesting to note that the work done by each steam drill has been greatly increased by the simple expedient of measuring the number of feet drilled daily by each machine. It is the sole duty of one man to keep a record of the work done by each driller. Before this method of individual record keeping was adopted, the drills averaged about 38 ft. of hole per 8-hour day per drill. Now the average is close to 57 ft. per day, showing an increased efficiency approximating 50 per cent. The drills are 3⅝-in. Rand drills, and the average depth of holes has been approximately 12 ft. The rock is gneiss and mica schist usually very hard, but occasionally "rotten" in spots, causing some delays, due to the sticking of bits. A handful of broken cast-iron (size of walnuts) thrown into a drill hole usually stops the sticking of bits. Of course the bit is removed before applying the cast-iron remedy.

As above stated, dump wagons are used to convey the earth to the dock, which is nearly a mile distant. These dump wagons average about 2 cu. yds. of earth, place measure, per load, holding, as they do, one full 2½ cu. yd. bucket of loose earth. Plank roads have been laid in the pit, but so much earth falls from the wagons upon the plank that three-horse snatch teams are needed to pull the loaded wagons away from the shovels and out to the street.

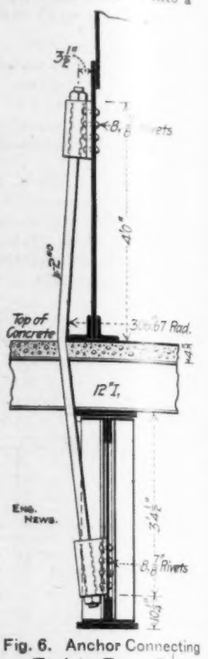


Fig. 6. Anchor Connecting Tank to Tower Framework.

One more point regarding the shovel work is worthy of attention. When a shovel reaches the end of its run, it must, of course, be shifted back to start a new swath or slice. At first this shifting was done in the usual manner by using short 5 ft. lengths of track, carried from the rear to the front of the shovel as it progressed. In this way, a night shift of seven or eight hours was required to move about 700 ft. By the method now employed a shovel is moved 700 ft. in two hours. This method consists simply in having a track laid the full length of the cut, so that, after shifting the shovel on to this track, it travels rapidly back to the starting point, losing in all not over two hours of working time.

While the work is progressing at good rate now, obviously it will be facilitated when cars replace

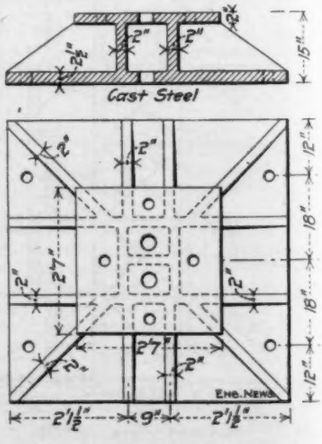


Fig. 4. Cast-Steel Column Bases, East Providence Water Tower.

wagons for conveying the material, and this is particularly true of the rock part of the excavation.

The contractors for this work are the New York Contracting & Trucking Co., whose chief engineer is Mr. R. G. Collins.

*See Engineering News, June 2, 1904, for plans showing location and character of this work.

THE WASHINGTON TERMINAL IMPROVEMENTS OF THE BALTIMORE & OHIO AND THE PENNSYLVANIA RAILWAYS.

By W. F. Strouse.*

After years of study on the various schemes for the elimination of grade crossings, in the City of Washington, a plan has been decided upon, along

Fig. 7. View of East Providence Water Tank and Tower.

the lines of which construction is well under way.

In February, 1901, Congress passed two acts which were supposed to be the final solution of the railroad problem in Washington. One authorized the construction of a new freight and passenger terminal for the Baltimore & Ohio R. R. at Delaware Ave. and C St., N. E., while the other authorized the enlargement of the present terminal of the Pennsylvania R. R. at 6th and B Sts., N. W., and provided for the correction of alinement on Virginia Ave.

Soon after the passage of the above acts, the American Institute of Architects appointed a commission to prepare a plan for beautifying and developing the City of Washington and its outlying parks. This commission consisted of Messrs. D. H. Burnham, C. F. McKim, Augustus St. Gaudens and Fred. Law Olmsted, Jr. Its work was barely begun before it became evident that a proper treatment of the park system required the removal of railway tracks and stations from the sites authorized by Congress. The site upon which the Baltimore & Ohio R. R. Co. was authorized to erect its terminal placed a busy commercial center within a square of the Capitol grounds, while the Pennsylvania R. R. Co.'s occupation of the Mall forever blocked any adequate treatment between the Monument and the Capitol.

*Assistant Engineer Washington Terminal Co., Washington, D. C. In charge of B. & O. R. R. Co.'s portion of improvement.

These facts led to the introduction of a bill providing for the erection of a Union Station, of monumental design, and the construction of the necessary approaches to it. This bill, embodying most of the provisions contained in the acts of 1901, but so modified as to meet the requirements of the two companies interested, was passed in February, 1903.

APPROACHES TO STATION.—The site selected for the terminal depot was the low section of ground at the intersection of Massachusetts and Delaware Aves., the average elevation of which was about 22 ft. above mean tide.

The northern approach selected was that decided upon by the Baltimore & Ohio Co., under the act of 1901, which occupied the bed of Delaware Ave. to Florida Ave., at which point the Washington Branch connection curved to the right, occupying a line parallel to New York Ave. and 100 ft. northwest of it; while the Metropolitan Branch connection continued in a northerly direction to a connection near South Brookland. From the junction with the Washington Branch, near Winthrop Heights, the line of the Pennsylvania continues in an easterly direction to a connection with its present line at Magruder. It will be observed from the general plan, Fig. 1, that this will require the abandonment of the present Washington Branch to a point near Langdon, and the Metropolitan Branch to South Brookland, a distance of about three miles in each case.

The southern approach will be through a tunnel under 1st St., east, between the Capitol and the Congressional Library to New Jersey Ave. and D St., S. E., and thence by open cut and viaduct southwesterly to a connection with the present Pennsylvania line on Virginia Ave. near Second St., west.

STATION PLANS.—The elevation of the floor of the station was fixed at 58 ft. above mean tide. This involved some serious changes in street grades. To simplify to some extent these changes, a grand plaza, 500 ft. wide by about 1,000 ft. long, decorated with balustrades, terraces and fountains, will be formed in front of the building. Into this space will lead nine separate and distinct streets, three of which are new and two of which are old streets modified better to fit the plaza scheme. To form the plaza and bring the streets to the proposed new level will require about 750,000 cu. yds. of earth; the fill over a considerable area being 35 ft. deep.

Through these radiating streets and avenues large crowds of people can leave or enter the station without difficulty, while the plaza will provide ample space for massing troops and spectators for such public ceremonials as take place at the Nation's Capitol.

The station faces toward the dome of the Capitol, less than half a mile distant, and its architectural treatment is in keeping with its character as the vestibule of the Capitol. As so many of the public buildings of Washington have taken their architectural inspiration from Greece and Rome, a freely interpreted classic may be said to have become the recognized architecture for the public buildings of Washington. Taking this view of the function of the Union Station as the one gateway of the Capitol, its architectural motives have been drawn from the triumphal arches of Rome.

The head house, shown in plan by Fig. 2, is 620 ft. long and from 65 to 120 ft. in height, and is being built of white granite. The three entrance arches, 50 ft. in height and 30 ft. in width, far exceed in scale their Roman prototypes. These central doorways lead into a vaulted open-air vestibule and thence into the main waiting room. At the end pavilions there are 40-ft. arch carriage entrances, the east one leading to a suite of apartments for the president and guests of the Nation; the west one leading to a general carriage porch near the ticket and baggage lobby. The central vestibule and the end pavilions are connected by an open-air portico, which, with the vestibule, constitute a continuous covered porch along the front of the entire building.

The general waiting room, 130 ft. wide by 220 ft. long, is covered by a Roman barrel vault, 90 ft. high, decorated with sunken coffers or panels after the manner of the bath of Diocletian It is lighted by a semicircular window 75 ft. in diameter at each end, and by five semicircular windows of 30 ft. diameter on each side. At the east end of this hall are grouped the dining room, lunch room and women's waiting room; at the west end, on opposite sides of a lobby 50 ft. wide, are located the ticket offices and baggage checking room, adjacent to which are the smoking room and package room; while telephone and telegraph booths will be located within the limits of the general waiting room. The baggage room is located in the basement, access to which will be had at the west side of the station. To avoid conflict between passengers and baggage on train platforms, certain platforms are set aside exclusively for baggage.

The passenger concourse or lobby is 130 ft. wide by 760 ft. long, covered by an arched ceiling in a single span decorated with panels, part of which will transmit the light. This space far exceeds in size anything ever built for a similar purpose. The concourse is divided by the usual fence, allowing a width of about 80 ft. for handling out-

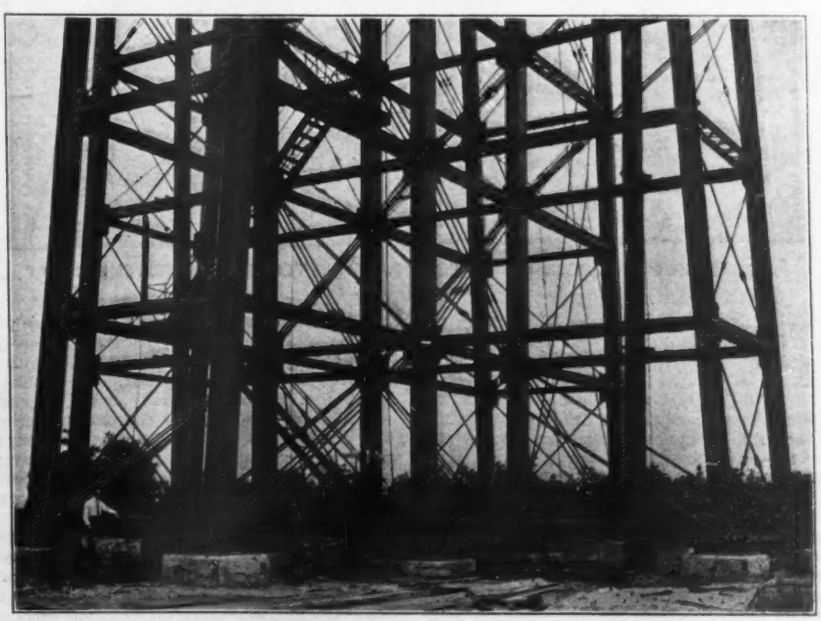


FIG. 8. VIEW SHOWING DETAILS OF BASE OF EAST PROVIDENCE WATER TOWER.

going passengers, while the remaining 50 ft. between the fence and ends of tracks will suffice for handling passengers arriving on trains.

The width of the train yard is 760 ft., one-half of which is located on either side of the center line of Delaware Ave., which was taken as the axis of the terminal. Of this width, the east 280 ft. will be used to accommodate the business handled over the southern connection, via the tunnel under First St., the plaza and the station. The remaining 480 ft., which will be on a higher level, will have tracks arranged for handling the business of both companies between Washington and New York, and the business of the Baltimore & Ohio R. R. Co. between Washington and points West.

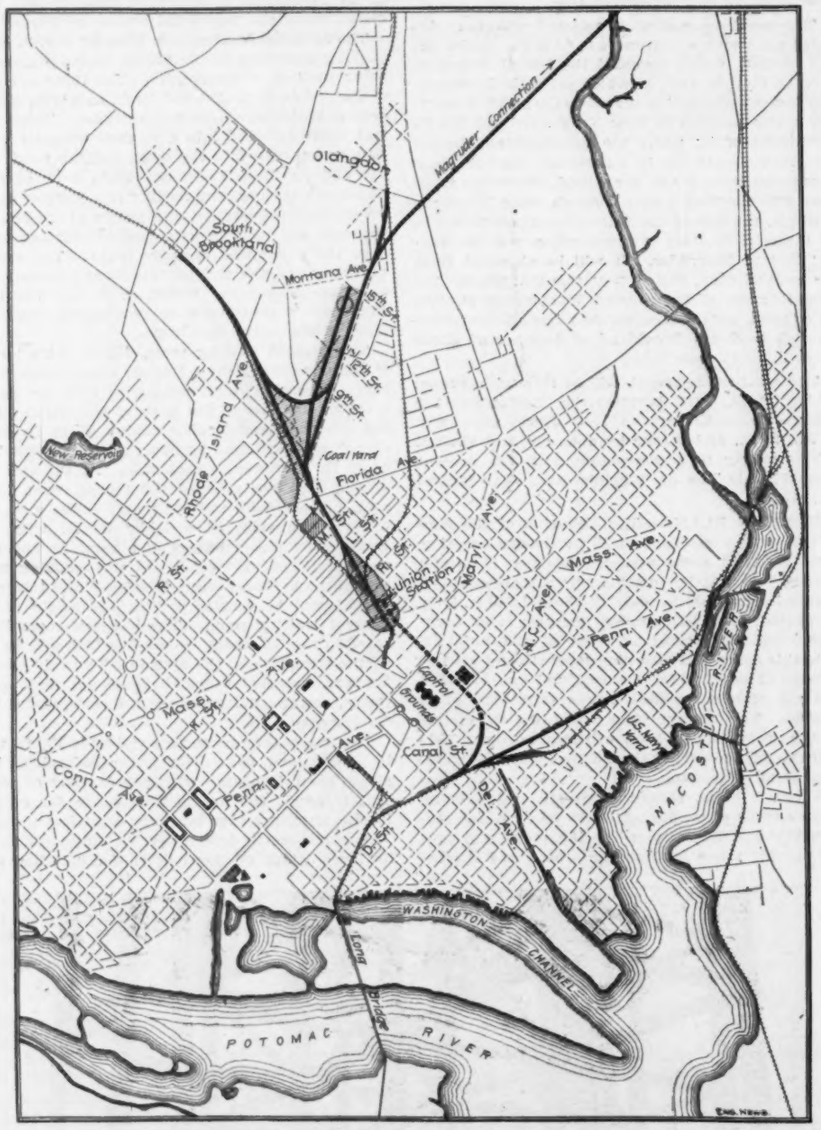


FIG. 1. SKETCH MAP SHOWING UNION RAILWAY TERMINAL IMPROVEMENTS AT WASHINGTON, D. C.

PREPARING THE SITE.—On the date of the passage of the Union Station Act, the space which will be occupied by the terminal was covered with dwellings, warehouses, coal yards, freight yards and sheds and the main tracks and sidings of the Baltimore & Ohio R. R. Co.'s Washington and Metropolitan Branches. Within eight or ten months after the passage of the act, all buildings were removed preparatory to starting construction. The coal and freight yards could not be removed until facilities had been provided elsewhere for this branch of the business, which was no small task.

With the removal of the coal and freight yards to their new location, the passenger tracks were thrown to the west side of the terminal to permit, so far as possible, the completion of masonry and filling within the terminal limits. This also made it possible to depress H St., which was crossed by both the Washington and Metropolitan Branch tracks, about 250 ft. apart, and occupied by a double track underground trolley line.

Attention having been called to the surface obstacles, we must not overlook those beneath the surface. A plan of the underground construction shows trunk line sewers and drains, gas mains and service pipes, water mains and service pipes, electric conduits for light, power, telephone and telegraph wires; all of which interfered with the progress of the work. A 9-ft. 6-in. trunk line sewer on F St. ran diagonally across the station site. Its removal required the construction of a new sewer about 1,400 ft. long. An intercepting sewer was constructed outside the east terminal wall between H and F streets, while new sewers and drains were constructed on all streets whose grades were materially changed.

The most serious underground obstruction, however, was a 15-ft. × 17½-ft. trunk line sewer on Florida Ave. When the new grade of this avenue was arranged it was found that the crown of this sewer would come above the proposed surface of this street. To avoid reconstructing this sewer, the grade of the terminal tracks from K St. northward was increased from 0.90 to 0.95 and a shallow system of bridge construction adopted for carrying the tracks over Florida Ave.

South of F St. a 3-ft. brick sewer in the bed of First St. had to be rebuilt, between F and G streets, N. E., for a distance of 700 ft., on account of the construction of First St. Tunnel; and a 4-ft. 6-in. sewer on D St., S. E., had to be rebuilt a distance of 400 ft. to pass under the tunnel at that point. A 24-in. gas main on F and First streets required a relocation of about 4,200 ft., while a 7-in. main on G St. required a change of about 2,700 ft. All the service pipes on depressed streets were disturbed. The change in gas pipes south of the Terminal Station, outside of First St., above mentioned, were of little consequence. A 24-in. water main on K St. had to be lowered, between First and Third streets, about 13 ft. at the deepest point.

DIVISION OF CONSTRUCTION WORK.—For purposes of construction, Massachusetts Ave. was made the dividing line, the Pennsylvania R. R. taking charge of all work south, the Baltimore & Ohio R. R. all work north, except the Pennsylvania R. R. connection between Winthrop Heights and Magruder.

On June 4, 1903, the Baltimore & Ohio Co. awarded Stewart & Wilmoth the contract for grading the coal yard site, estimated to require the removal of 100,000 cu yds. of material; and the bulk freight yard, at New York and Florida avenues, estimated at 50,000 cu. yds. On July 21, 1903, the contract for the graduation and masonry of the northern approach was let to McMullen & McDermott, and the Hoffman Engineering & Contracting Co The estimated quantities of this contract were: Excavation, 2,600,000 cu. yds., and masonry, 100,000 cu. yds. On Oct. 10, 1903, the contract for the construction of the Terminal Station, covering the foundations and superstructure, with the exception of the structural steelwork and interior finish, was let to the Thompson-Starrett Co., of New York. On Dec. 10, 1903, the Pennsylvania Co. made a contract with the New York Continental Jewell Filtration Co., covering the construction of the tunnel under First St., from D St., N. E., to the south portal at New Jersey Ave. and D St., S. E.

About 2,250,000 cubic yards of the excavation covered by the McMullen & McDermott and the Hoffman Engineering & Contracting Co.'s contract was located on the space to be occupied by the coach yard, roundhouse and shops, north of Florida Ave. About 90% of the masonry is located between Florida Ave. and the station, consisting of retaining walls on the east and west sides of the terminal and between the high and low tracks, and abutment walls along the north and south lines of all depressed streets. Nearly 1,000,000 cu. yds. of material will be required to fill the space within the limits of the terminal property, between Massachusetts Ave. and L St. This material can not be deposited in any considerable quantity until the masonry walls are constructed. This applies as well to the filling necessary around and between the foundations of the station building. An average fill of 20 ft. will be required over the entire area occupied by the building, to provide a level for the basement floor, which is fixed at an elevation of 42 ft. Owing to the great height and small sectional area of some of the station piers, great care is necessary in depositing material to avoid injury to the masonry.

ARRANGEMENT OF STREET CROSSINGS.—The grade line very closely coincides with the original ground line between K St. and Florida Ave. To take H, K, L and M streets and Florida Ave. under the tracks of the railroads, it was necessary to depress them from 11 to 16 ft. below the former grades at the deepest points, which, in order to avoid excessive grades, required the change to extend some distance beyond the limits of the terminal occupation except at the west end of H St. subway. The treatment of avenues and streets north of Florida Ave. will be as follows: Rhode Island Ave. will be taken under; New York Ave., 9th, 12th and 15th streets will be taken over, and while it is not definitely decided, Montana Ave. will probably be taken over. South of the station all streets will be taken over to and

including New Jersey Ave., from which point to connection on Virginia Ave. they will be taken under.

RETAINING WALLS.—The east and west retaining walls, defining the terminal limits from the depot building to M St., range in height from 5 to 35 ft above the present surface of the ground, the greatest height obtained being on the west side, just north of the station. All exposed masonry on the improvement is being constructed of sand stone facing, backed with Portland cement concrete, the stonework in all cases being carried 12 to 18 ins. below the finished surface. All foundations are constructed of Portland cement concrete.

All retaining walls, except the east wall between H. and K. streets and the west wall between the station concourse and the power house, are of the ordinary section and constructed in the usual manner, Fig. 3. The east wall, between H and K streets, required a special design on account of the functions it had to perform. At I St. an entrance to the express building at the track level is provided. At H St. there is an entrance to the basement of the express building, 18 ft. below. The grade of Second St., between these streets, is about 4%. The wall was designed for retaining wall at I St., while at H. St. it was designed to carry the girder work of the upper driveway, and is only 3 ft. thick above the foundation. The sections between the above points vary to suit the conditions.

The west wall, above excepted, also has a dual mission to perform. As the station and heating plant are separated by about 1,000 ft. it was necessary to provide a pipe and wire subway between them. To construct a separate subway between these points involved a heavy expenditure, owing to the 35-ft. fill above and the bad foundations below the present surface. As a matter of economy it was decided to build a subway in the retaining wall, through which these pipes and wires would be carried. At H St. it was necessary to depress the subway so as to pass under the tracks of the underground trolley road, while the pipes and wires leading to the express building will be located in a smaller subway under the north sidewalk of H St.

The pipes will be supported on pipe racks, suspended from I-beams in the ceilings of the subways, while the wires will be carried in conduits, along the sides of the subways.

Considerable attention has been given to the question of drainage. Gutters are being constructed in the backs of all street bridge abutments, with down spouts at intervals, to be connected with sewers in the streets below. The backs of the abutments are being covered with a ½-in. layer of waterproofing, applied at boiling temperature upon a thoroughly dry surface. In this way it is believed the masonry will be kept dry.

BRIDGES OVER STREETS.—The bridges which will carry the tracks over the streets are of unique construction, Fig. 4. The distance between abutments in all cases is 80 ft. at right angles to the axis of the street, divided into three spans; the center span being 25 ft. and the side spans 27 ft. 6 ins. As the tracks will cross the streets on a skew of 15° 42′, the above spans will be slightly increased, as the beams will lie parallel to the tracks.

In addition to the abutments, these bridges will be supported on two lines of columns, which form the center span. The superstructure will consist of 24-in. I-beams, spaced about 18 ins. centers, embedded in concrete divided into two sections, Fig. 5; one extending from 2 ins. below the bottom of the beams to the top of the beams, upon which will be placed a layer of waterproofing ¾-in. in thickness. The other section will consist of reinforced concrete, about 6 ins. thick, placed on top of the waterproofing. The introduction of metal in the top section of concrete is for the purpose of preventing checking and cracking. It is thought this will furnish a dry structure, as it will be noted that the waterproofing is carried down to the gutter above mentioned, thus completely covering the top of the bridge and the backs of both abutments.

As the shortest of these bridges will carry ten tracks, divded into two groups of five each, it is proposed to provide for expansion and contraction by introducing vertical joints (Fig. 5) at intervals of about 30 ft. These joints will be thoroughly waterproofed, and to further insure against dripping to the streets below, additional drainage appliances will be provided where necessary.

As the shortest subway will be about 135 ft. long and the longest about 790 ft., it was considered necessary to provide some method for lighting these subways (Fig. 6), and some attention has also been given to the question of ventilation. At Florida Ave. and M St. provision is made for prismatic lights, covering an area of 3 ft. by 80

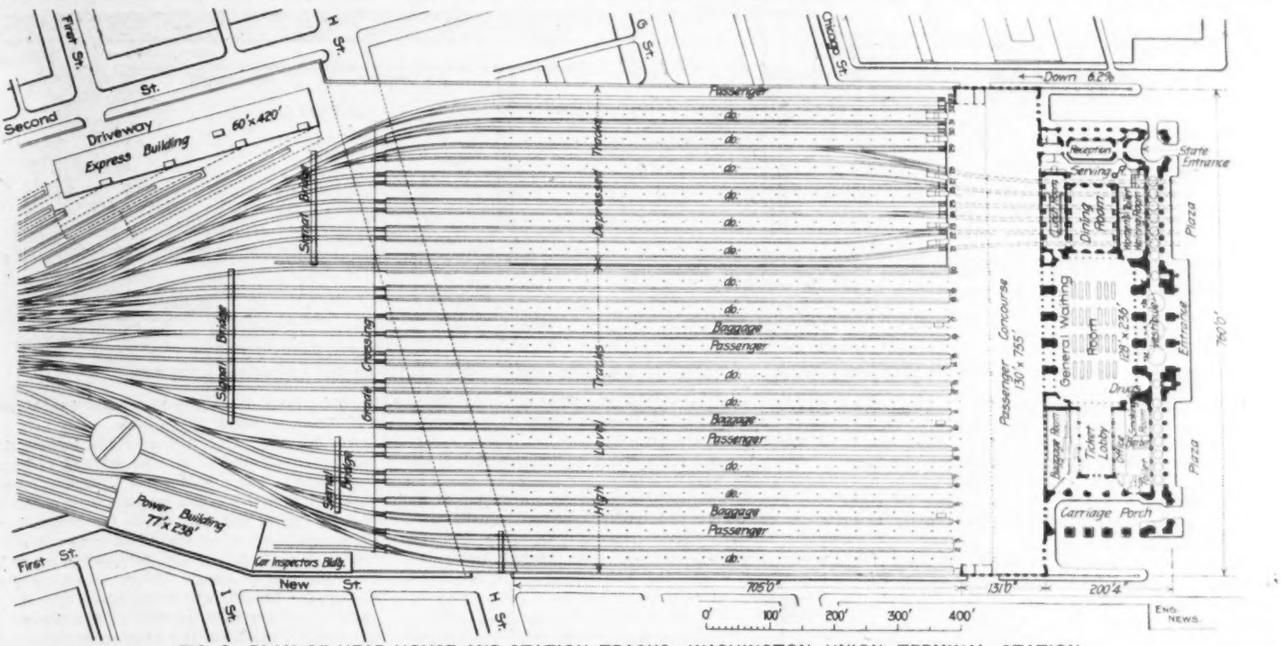


FIG. 2. PLAN OF HEAD-HOUSE AND STATION TRACKS, WASHINGTON UNION TERMINAL STATION.

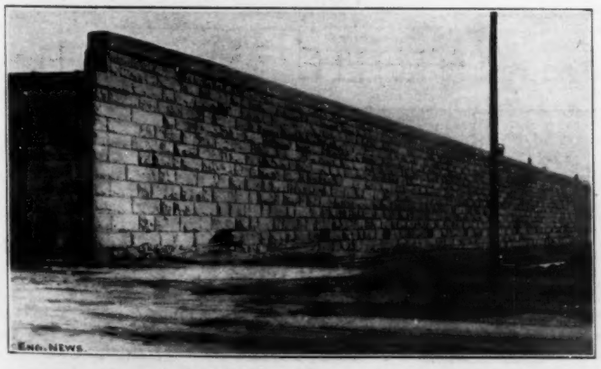


FIG. 3. VIEW OF COMPLETED RETAINING WALL, WASHINGTON UNION TERMINAL STATION.

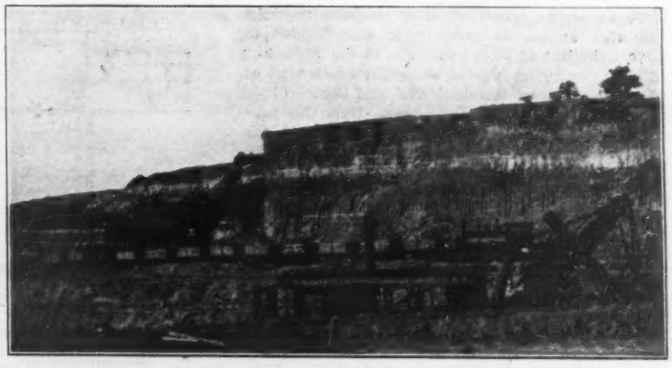


FIG. 9. VIEW SHOWING STEAM SHOVELS AT WORK ON EXCAVATION FOR PASSENGER YARD.

ft. in each case. At L St., where the length is somewhat greater, an opening about 10 ft. by 80 ft. is left about the center of the structure for light and ventilation. At K St. an opening is provided about 10 ft. by 80 ft., near the center, with two prismatic light spaces on each side, 3 ft. by 80 ft. At H St., the longest structure, prismatic lights covering an area of 17 ft. by 80 ft. are provided for in the passenger platform to the number of nine. Between the high and low levels an opening will be left for ventilation. This space will be about 3 ft. by 80 ft. It is believed by this method that both light and ventilation will be obtained to such a degree that the objectionable features of such structures will be greatly lessened, if not obviated.

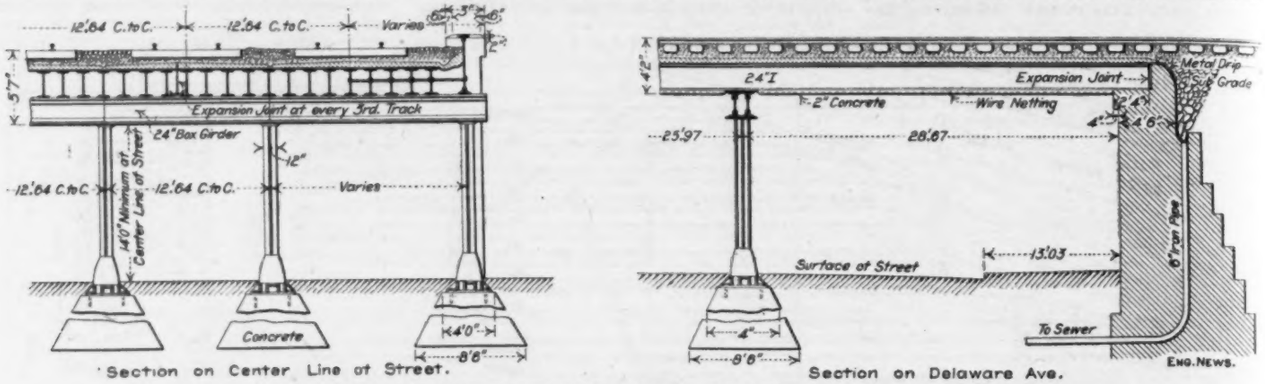


FIG. 4. SECTIONS SHOWING CONSTRUCTION OF BRIDGES CARRYING TRACKS OVER CROSSING STREETS.

The abutment walls (Fig. 4), constructed for carrying the above mentioned bridge superstructures, are of the ordinary construction, except the change in section to provide for drainage, and the construction of manholes for power, telegraph, telephone and signal conduits.

WIRE GALLERIES.—To provide for all wires between R St. and the station, a system of conduits has been arranged between the tracks; north and east of R St. they will be carried on poles.

The conduits will be laid in concrete cradles, in the usual manner, at all points except through the bridges, where they will be placed between the I-beams. Owing to the difference in elevation of conduits, through the bridges and between them, it was necessary to construct manholes in each abutment and at intermediate points when block distances were excessive.

To avoid the construction of conduits on filled material, as far as possible, it was decided to deflect the line of power conduits at the south line of K St., carrying it west along the south wall of K St. to the west terminal wall and thence along the west wall to the power house. Over this portion of the line the conduits will be laid in the concrete backing. From K St. to the station the telegraph, telephone and signal wires will be carried in fiber ducts (except through H St. bridge) laid in wooden boxes until settlement has taken place, when permanent conduits will be put down.

RETAINING WALL CONSTRUCTION.—The construction of the long retaining and abutment walls found on this work brought out methods not employed on the average piece of construction. The presence of so many railroad tracks and yards, located in the very heart of this construction, also developed some unusual methods. This was true also in connection with the foundation construction for the depot building, where so many piers of all sizes had to be constructed to a height of from 25 to 35 ft. above the ground level.

Preparatory to starting the construction of masonry, a large cubical concrete mixer was installed near the intersection of the east wall and G St. A pit about 16 ft. wide and 40 ft. long was constructed about 100 ft. from the mixer. Across this pit were laid two surface material tracks, which extended a sufficient distance beyond the pit to provide storage for five or six cars each; one track was used for sand and the other for crushed stone. Between these tracks was located the cement house, with one end extending to within a few feet of the pit. The cars containing sand and stone were dropped down by gravity to a position over the pit.

A truck, used for conveying the material to the mixer, was first placed under the sand car, and the proper quantity of sand was dumped direct from the car. The truck was then moved to a point midway between the tracks, where the cement is added. It was then moved under the stone car, where the requisite quantity of stone was taken direct from the car. The truck was then taken up an incline by a cable and automatically dumped into a large hopper above the mixer, after which it returned to the pit, while the material just deposited was dropped into the mixer. The water was added by an automatic device, which, under normal conditions, gave very uniform results. When sufficiently mixed it was dumped into one of several forms of buckets, on a small truck, and conveyed by horses or mules to the point where it was to be used.

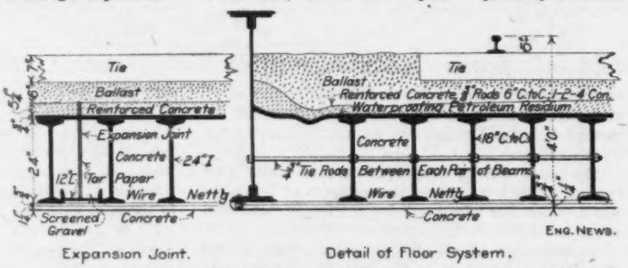


FIG. 5. DETAIL OF SOLID FLOOR CONSTRUCTION FOR OVERHEAD BRIDGES.

This plan worked satisfactorily as long as the shipments of materials and the progress of the work ran approximately together. In case of bad weather or any delay on the work, which would bring together more material than could be [mixed] for within the time limit for releasing the cars, the contractors were confronted with demurrage charges. Owing to the simplicity of the method and consequent cheapness of handling the material, a reasonable amount of demurrage could be afforded. The greatest difficulty was experienced in mid-winter when the very cold weather set in and masonry could not be laid.

At the mixer installed near the intersection of the west wall and G St. the method of handling the material was somewhat different. An elevated double track line of bins was constructed, one for sand and the other for crushed stone. Cars containing these materials were run over the bins, dumped, and the cars released in a few hours after arrival. A cement house was located adjacent to the bins in which the stone was stored in such a position that, to unload the cement from the cars on the bins, the bags were thrown down a chute through the roof of the house.

A subway about 6 ft. square was built under each line of bins. The truck used for conveying material on returning from the mixer first passed through the subway under the sand bins, where sufficient sand for a batch was taken from the bins by opening an aperture in the bottom by a simple lever device. The truck was then moved to the outer end of the bins to a transfer table, so arranged that its action was practically automatic. It was then moved laterally to the track under the stone bins, where the stone was taken in the same manner as the sand above described. At the end of the bin near the mixer a hopper was arranged into which cement was placed in batch quantities. As the truck passed this point the cement was added. The truck was then taken up an incline by cable and dumped into a hopper over the mixer, as above described. The quantities of materials were gaged by metal strips riv-

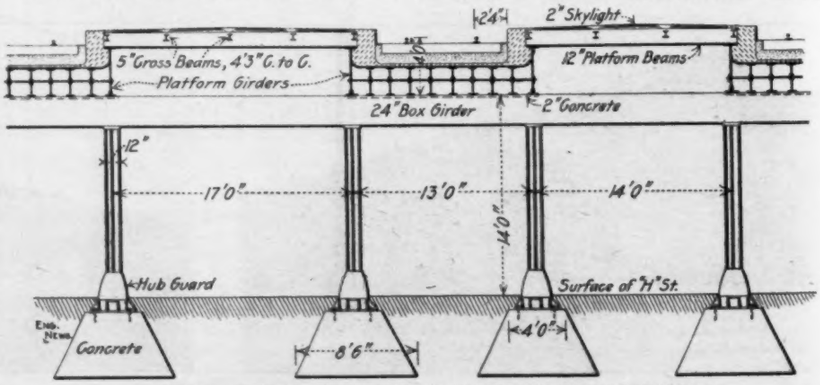


FIG. 6. DETAILS OF LIGHTS IN OVERHEAD BRIDGE FLOORS.

etted to the sides of the trucks, placed in position from measurements made by the engineer.

A Hains' gravity mixer was installed in the bed of L St. The stone and sand used at this mixer were taken direct from drop bottom cars, placed in position on a trestle over L St., which had previously been depressed to the new grade.

This material was dumped through the floor of the trestle into buckets, in which it was hoisted to the top platform of the mixer. The cement which was housed nearby was taken to the top of the mixer in the same manner.

Another of these mixers was located within the limits of the station building and used to mix the concrete used in the foundations of this massive structure. The method of handling the crushed stone in this portion of the work was different from any above described. In this case the stone and sand were dumped from cars into bins under an elevated trestle. The bottoms of these bins were arranged as chutes, which were opened and closed by a form of cut-off, raised and lowered by hand. Certain bins were used for sand, others for stone. These materials were taken from the chutes into buckets, which were hoisted to the top of the mixer by derricks. The cement house was so located that the bags were run down a chute from the cars on the trestle through the roof of the house.

As previously noted, the floor of the station was fixed at an elevation of 58 ft. This required the construction of foundations, which extend from 30 to 35 ft. above the ground, requiring a most elaborate and intricate system of forms for molding the concrete to the proper shape.

Regarding the merits of the different mixers on this work, it may be said that, for thorough mixing, nothing is better than a rotary, whether it be the cubical, the Smith or the Ransome patterns. For speed and good average results, the Hains is commended, but its method of mixing does not permit mixing beyond a certain limit.

As so much of the work consisted of long stretches of wall, traveling derricks were used in most cases, and where possible one rail of the derrick track was used as one rail of the track upon which the masonry material was handled. When the concrete foundations were in, the stone and concrete work were put up as follows: A course of stone facing was laid, after which the concrete backing was rammed in place, in layers about 8 ins. thick, each course being completed and allowed to set before another course was started. On a work of this size, this method worked well and gave the concrete time to get hard before it was necessary to start the next course.

The forms used in putting up the concrete work were of a very simple design. Small iron rods, with one end supplied with a nut and the other with a hook about 2 ins. long, were set in the top of each course, for the purpose of holding the forms for the next course in place. When the concrete was set these rods were cut off.

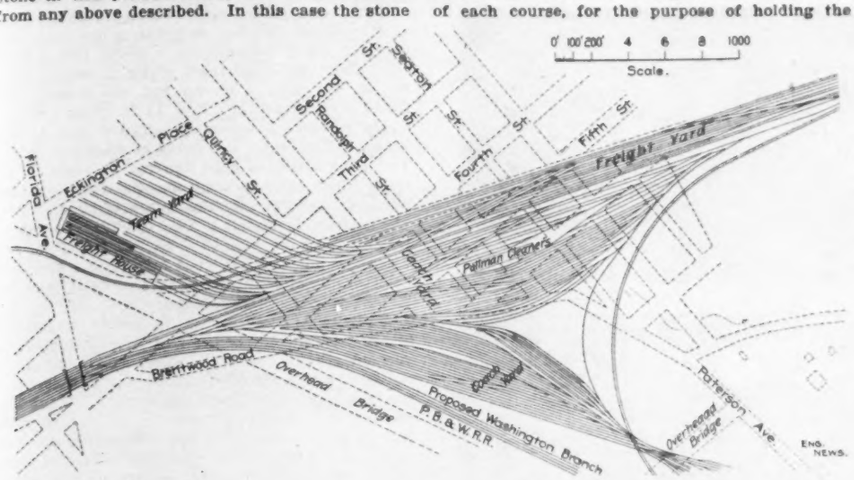


FIG. 7. TRACK LAYOUT OF NEW FREIGHT TERMINAL, WASHINGTON, D. C.

BALTIMORE & OHIO COAL YARDS.—As the space originally used by the Baltimore & Ohio R. R. Co. for coal and team yards will be occupied by the Terminal Depot, it was necessary to provide facilities at some other point for this branch of the business. The site selected for the new coal yard was located between First and Second and M and N streets (Fig. 1). In this space six lines of trestle, about 600 ft. in length each and 18 ft. in height, were constructed, having a capacity in all of about 60,000 tons. The driveways are 33 ft. in width, one for each trestle, and were macadamized. The bins cover three-fifths of the total area of the site, while the drives cover the remaining two-fifths. The trestles are all constructed level, but owing to the grade of First St. and of the terminal tracks on Delaware Ave., it was found necessary to construct the yard on a city and the good character of most construction make it a great market for all kinds of building material, which is handled in carload lots.

With these facts to work upon, team or bulk unloading tracks were provided to accommodate about 450 cars, while the house tracks will care for about 200 cars, with a freight storage yard of about 600 cars. The team and house tracks are divided into three groups, each having a switching track, of some length, which will permit several yard engines to work the groups without interference. The storage yard is divided into three sections, so arranged that the switching can be done with the least interference.

PASSENGER YARDS AND SHOPS.—East of the freight terminal is located the joint passenger coach yard, roundhouse, shops, engine yards and repair tracks, Fig. 8. Two tracks, between the terminal train yard and the roundhouse, are set aside for handling engines and cars between the station and the coach yard, and roundhouse, without interfering with the movement of trains over the main passenger tracks. They are located between the Washington and Metropolitan Branches, and pass under the Y connection of the above lines.

At Florida Avenue all tracks are at the same elevation. From this point north and eastward the grades of the groups of tracks are as follows: The Washington Branch connection of the Baltimore & Ohio R. R. and the Magruder connection of the Pennsylvania R. R. ascend at the rate of 0.9 per cent.; the coach yard tracks at 0.5 per cent., and the Metropolitan Branch connection of the Baltimore & Ohio R. R. at 0.7 per cent. These differences in grades are due largely to the natural contour of the country and served to permit an easy overhead crossing of the Y connection between the Metropolitan and Washington Branches over the equipment tracks, which are constructed on the same grade as the coach yard, southwest of the Y, but which have heavier grades north of it.

From a construction standpoint heavier grades would have been preferable on all the different groups of tracks, as about 2,250,000 cu. yds. of earth will have to be removed to bring the space assigned as a passenger coach storage yard to the proper grade. Fig. 9 is a view showing steam shovels at work on this excavation.

A better idea of the size of this task may be had, when it is stated that the material, over a considerable area, has to be removed to a depth of 82 ft., and that about 75 acres of ground will be occupied for this branch of the business. Six large steam shovels, each having a capacity of about 30,000 cu. yds. per month, are engaged on the work and to date have moved about 1,000,000 cu. yds. of material, practically all of which has been wasted. Quite a large percentage of this material was hauled nearly two miles from the point where it was excavated. Practically all of the remaining material will be used to fill in the terminal space and the plaza. Should there be any material left after this work is done, it will be used to fill property now owned by the railroad company around the plaza, which will be 30 to 35 ft. above the present ground level.

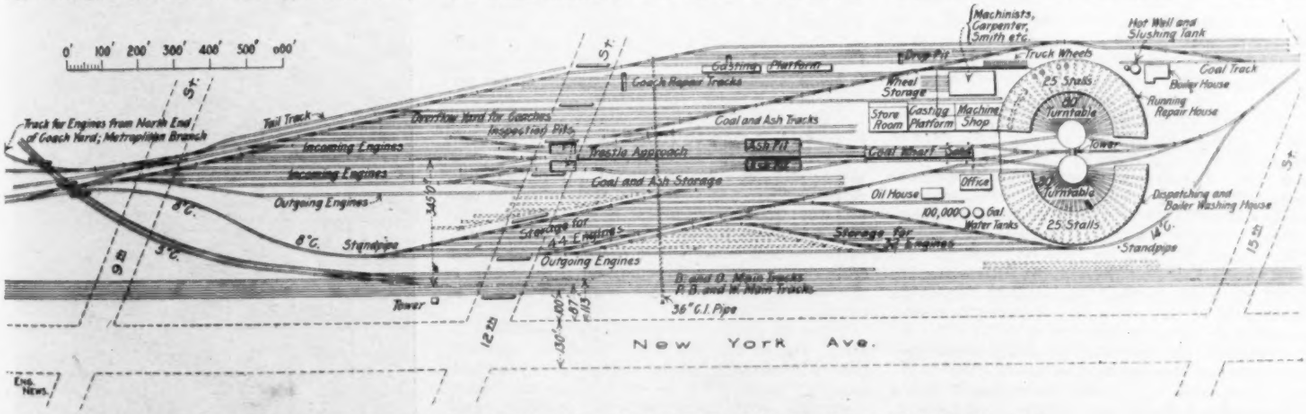


FIG. 8. PLAN OF PASSENGER YARDS AND ROUNDHOUSE PLANT, WASHINGTON, D. C.

As designed, the coach yard will have a capacity of at least 600 cars. Considerable thought has been given the roundhouse and shop layout, and the arrangement will be quite convenient and complete. The engine house, it will be noted, will be constructed in two sections of 25 stalls each, both sections having a turn-table. The coaling plant will consist of storage bins with measuring devices, and is located between the ash pits and the roundhouse. The approach to the storage bins will be by the ordinary trestle incline. The coal will be delivered in the bins direct from drop bottom cars, the cost of handling being practically nothing. This style of coal wharf requires more room than a mechanical device, but is cheaper both in first cost and operation.

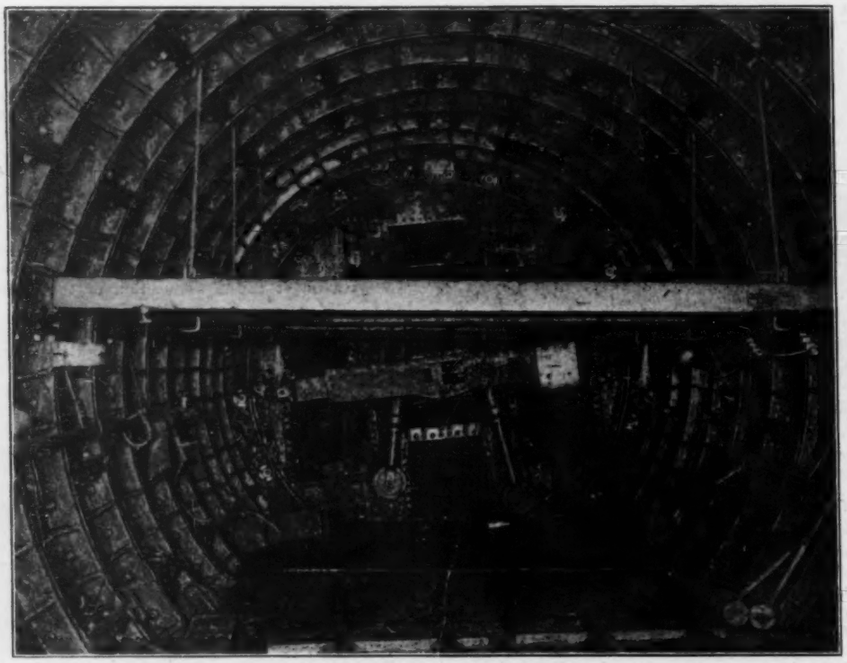


FIG. 1. SHIELD IN THE SOUTH TUBE OF THE HUDSON RIVER TUNNEL.

The general construction of the ash pits will be very similar to ordinary practice, but the ash track will be depressed sufficiently to permit loading the cars by shoveling the ashes over an adjustable apron, the top of the car being below the bottom of the engine pit. Inspection pits are also provided and are reached just before going on the ash pit, where the engines can be examined before turning them into the house.

The track arrangement will be such that access to both tables can be had from both sides of the coal wharf and from both tables to the storage yard. A tail track is provided for moving engines to the storage yard without fouling inbound tracks. Shop facilities will be provided for making light repairs to cars and engines, and necessary tracks, platforms, etc., will be installed.

PENNSYLVANIA TUNNEL APPROACH.—As stated above, the southern approach to the Union Station will be through a tunnel under First St., the southern portal being located at New Jersey Ave. and D. St., southeast, and the north end terminating in the basement of the station building. It will be constructed for a double track line and will consist of two single track tunnels, built side by side, separated by a wall 4 ft. thick, Fig. 9. Each tunnel will have a width of 16 ft. and a height of 17 ft. from top of rail to soffit of arch. The center wall is being constructed with openings or passage-ways, located at intervals of 100 ft. to provide refuge niches for employees and means of communication between the tunnels.

The side walls and haunches are being constructed of concrete, the arches of brick and the center wall of stone. Terra cotta conduits are being laid in the haunches for the accommodation of wires; access to which will be had by manholes constructed in the side walls, above the spring line, at intervals of 400 ft. This section will be used from the south portal to a point 525 ft. south of the station building, a distance of 4,033 ft. From this point, to the south side of the station building, the construction will be girder covered way; the number of tracks covered ranging from two to six.

The top of the tunnel will be covered with four layers of waterproof felt, thoroughly coated with waterproof compound. The waterproofing will be protected by a 1-in. layer of cement, in the cut and cover portion, while the drift portion will be protected by a layer of brick. Gutters will be formed along the top, on each side, filled with broken stone. Drain pipes, connecting with these gutters and leading to subgrade, will be built in the side walls at intervals of 50 ft.

All work completed to date has been done in open cut. For that portion of the work where it will be found necessary or advisable to drift, it is proposed to carry the sidewalks far enough in advance to support a steel shield, which it is proposed to use for supporting the roof while the core is being removed and the center wall, arches and haunches are being constructed.

GENERAL REMARKS.—Respecting the work in general, it may be stated that, after the plans were matured, the construction, but for its magnitude, did not seem to present any serious difficulties. The most important matter at the outset was extreme accuracy in establishing base lines and bench marks. The peculiar track conditions required considerable care in fitting bridge seats to the grades—especially on H St., which is crossed by both the high and low groups of tracks.

The most serious and perplexing question to be dealt with, so far, has been difficult foundations. Generally speaking, all foundations have been bad, making it necessary to go much deeper than other conditions would have required. In most cases good gravel beds have been found at an elevation of from 9 to 12 ft. above tide.

When it is recalled that the elevation of the lowest ground in the locality is about 20 ft. above tide, some idea of the depths of foundations can be gotten. Conditions point to the conclusion that at one time much of the space being occupied by this terminal was low, swampy land, the general elevation being about 12 ft. above tide, and that a small stream ran diagonally across in the vicinity of G St.

In most cases good gravel is found about 2 ft. below the layer of black loam, which is thought to have been the original ground level. The worst conditions met were at K St., where tests showed nothing substantial at an elevation of about 10 ft. below tide, and pile foundations were resorted to.

Before arranging for pile foundations, however, a thorough examination of the material was made. The elevation of the ground, when work was started, was about 48 ft. above mean tide. From this elevation to about 20 ft. above mean tide the material was yellow clay containing thin layers of gravel; not forming, however, a material suitable for heavy foundations. Under this a blue clay, about the consistency of putty, which could be cut out with a shovel but not picked, was found for a depth of about 15 ft. Under this a sandy material, containing sufficient clay to endanger its stability, was found of uncertain depth. Test piles indicated good material at about 25 ft. below mean tide; showing unsatisfactory foundation conditions in a vertical range of nearly 75 ft. The nearest approach to this condition elsewhere was on the north side of H St., west of First St., where sand of a fairly good quality was found at an elevation of about 4 ft. above mean tide; about 25 ft. below the present street level.

Realizing the size of the undertaking, nothing but new machinery of the most improved pattern was brought to the work. No better plant could be desired than that owned by the various contractors engaged on this improvement. What has been said of the quality of the plant may with equal propriety be said of the contractors who are handling the work. The railroads interested have reason to congratulate themselves on the selection.

In the preparation of this article, I am indebted to Messrs. D. H. Burnham & Co., Architects, Chicago, for sketches and other information in regard to the station building; to Mr. Robt. Farnham, Jr., Assistant Engineer, P. B. & W. R. R., for information relative to construction of First St. tunnel, and to the members of my corps for photographs of the works.

THE REMARKABLE PROGRESS OF THE HUDSON RIVER TUNNEL FOR THE NEW YORK & NEW JERSEY R. R. CO.

To drive a submarine tunnel in river silt at the rate of 48 ft. per 24-hour day is a remarkable engineering achievement quite without precedent, so far as we are aware. When it is added that in doing this work not a car load of muck is removed through the tunnel, the wonder is not lessened, although, in fact, the secret of the unprecedentedly rapid progress is the freedom from the delays usually involved in excavating and tramming the excavated material.

Fig. 2. Curve of 150-ft. Radius in North Tube of the Hudson River Tunnel.

The tunnel to which we refer is that which is being driven underneath the Hudson River by the New York & New Jersey R. R. Co., about oppo-

site Christopher St., New York City. As most of our readers will remember, this is the old Hudson River tunnel enterprise, originally projected by the late De Witt C. Haskin and Trenor W. Park, and begun in the '70's and early '80's.

The work done then and at a later period, in 1889 and 1890, when an English company attempted to finish the work, consisted in the completion of one single-track tunnel nearly across the river. The new company which took hold of the enterprise last year, has completed the under-river section of this tunnel (the northerly one), and the work above referred to is that which is being done upon the south tunnel. The old company made only a bare beginning with the south tunnel; but the present company, although it only attached the south tunnel last March, has already pushed it out 2,500 ft. from the New Jersey shore, and the work has been at times interrupted by delays incident to acquiring terminal rights on the New York side. At present the air lock is located about 900 ft. back of the shield, thus leaving 1,600 ft. of open tunnel not under air pressure. This 1,600 ft. may be walked through without seeing so much as one drop of water falling from the cast iron lining; and, since no muck cars are being run, the floor of the tunnel is free from mud and perfectly dry.

As the reader will have inferred, the shield is pushed forward with closed doors, crowding aside and compressing the soft river mud in its pathway. To push forward the shield, which is 17 ft in diameter, 16 hydraulic jacks are used, as shown in the accompanying photograph, Fig. 1. Each of these jacks is 8 ins. in diameter and capable of operating under a pressure of 5,000 lbs. per sq. in., but the normal operating pressure is 3,500 lbs. per sq. in. During the visit of a member of our editorial staff, the shield was pushed forward 2 ft. in just 96 secs. The time required to put up and bolt the cast-iron ring of nine segments was about 35 minutes, thus making an advance of lined tunnel equivalent to 1 ft. in less than 20 minutes with a force of only 8 men in the face of the tunnel. This astonishing progress is not merely a display feat, for in each 8-hour shift the average advance is 16 ft., or eight complete rings 2 ft. long. The men actually work about 7¼ hours at the face, under a pressure of 26 lbs. per sq. in., and spend the remaining ¾ hour entering and leaving the work. The depth of the silt above the roof of the tunnel where the shield is now located is about 25 ft.

In this connection it should be noted that a physician is always in attendance at the mouth of the shaft, and his office is provided with a compressed air hospital to be used in case any man is attacked with "the bends."

The river mud through which the shield is now passing is about the consistency of plastic putty. It was this extreme softness that made contractors fearful of bidding upon the work. The engineer of the New York and New Jersey R. R. Co., Mr. Charles M. Jacobs, then attacked the work with a plant designed by himself and operated under his direction.

But an even more difficult feat, to our minds, in connection with this enterprise than tunneling beneath the river, is the tunneling for the approaches on the New York side. The approach for the north tunnel is now being driven. To understand the problem confronting the engineer, it must be remembered that the shield is being driven through loose sand at a depth of some 30 to 40 ft. below the street surface. The street is narrow and is lined with warehouses and other buildings all founded on sand far above the tunnel line, and sure to suffer settlement if in the driving of the tunnel sand is loosened beyond the perimeter of the shield.

The tunnel, moreover, makes a curve of 150 ft. radius and 90° of total curvature in passing from the shaft at the end of the river section at the foot of Morton St. to its terminal in the block bounded by Hudson, Christopher, West and Greenwich Sts. On this work the air pressure averages about 8 lbs. per sq. in., and sand is encountered instead of mud. The sand is so clean and coarse that it is sold for building purposes.

Progress in this sand is far slower than in the soft mud, for it is all excavated, and the face of the work is continuously supported with a plank sheathing. This sheathing is held by short braces that bear against the shield while excavation is in progress. When it becomes necessary to push the shield forward, long braces are inserted through the doors of the shield, these long braces conveying the thrust of the sand to the cast iron lining of the tunnel. As soon as the shield has been advanced 2 ft., these long temporary braces are removed, and short braces bearing against the shield are inserted.

Before the shield is pushed forward, a few sheeting planks are removed, and the sand is excavated for a depth of nearly 2 ft., in an annular ring which is then filled with clay. Upon pushing the shield forward, its hood or apron is forced into this clay effecting a seal that prevents the escape of compressed air.

Great care is exercised in grouting around the outline of the cast iron lining. A grout of neat cement is mixed in a small mechanical mixer especially designed by Mr. Jacobs for this work, and is forced out under a pressure of 150 lbs. per sq. in. through small plug holes in the iron lining. About 50 bbls. of cement are used daily in grouting. The roof of the tunnel is about 30 ft. below the surface of the ground.

Although the shield has passed under the corner of a large brick building, not the slightest settlement of the building has occurred. The success of the shield process under these conditions may be ascribed in general to three factors: (1) Extreme care in measuring and recording the elevation of each ring of the lining, so as to detect the slightest distortion; (2) the careful grouting of every space around the lining; and (3) the sheathing of the face of sand with plank, and the bracing in such a manner as to prevent any sudden caving either while the shield is still or in motion.

FIG. 3. FRONT VIEW OF SHIELD SHOWING CUTTING EDGES.

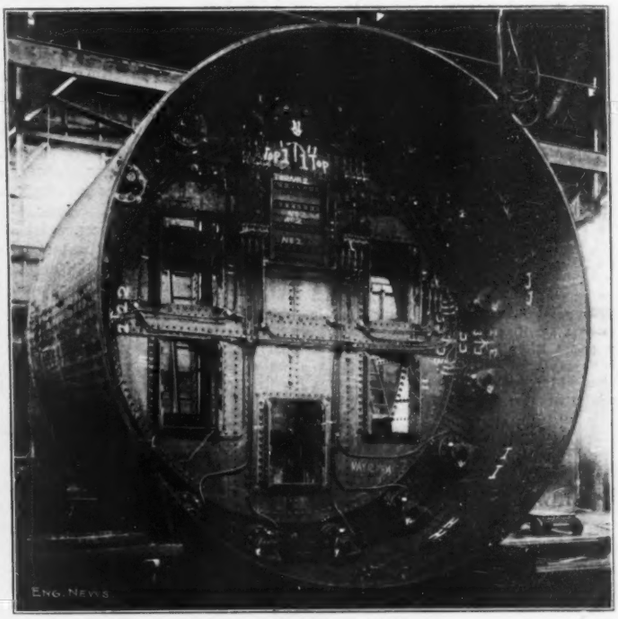


FIG. 4. REAR VIEW OF SHIELD SHOWING HYDRAULIC JACKS.

FIGS. 3 AND 4. VIEWS OF SHIELD FOR HUDSON RIVER TUNNEL UNDER CONSTRUCTION IN THE SHOP.

For courtesy in permitting us to study this work while in progress and for the photographs that illustrate it, we are indebted to Mr. Charles M. Jacobs.

THE WOOD WATER TANK AT UPPER SANDUSKY, O., which failed on Oct. 15 (see Engineering News, Oct. 27, p. 386), was banded with ⅝ × 4-in. steel, spaced 1 ft. apart. We are informed by Mr. Samuel J. Black, Superintendent of the Upper Sandusky Water-Works Co., that an examination of the bands shows that their outer sides were in good condition, and that their inner sides "were not injured enough to cause the bursting, except at the rivets, where the hoops were badly eaten." The tank was 25 ft. high, 25 ft. in diameter at the bottom, and 23 ft. at the top, and was erected in 1888. The brick tower, 100 ft. high, on which the tank rested does not appear to have been injured.

FORMULAS FOR RAPID BLUE-PRINTING PAPER.

The blue-printing process is so universally used and so well known that it may seem like going back to elementary things to print anything regarding the materials used in it. An inquiry has reached us, however, from a correspondent who asks for a formula for making blue-print paper for use in an electric printing frame, and it occurs to us that some of the present readers of Engineering News may be unfamiliar with a process for increasing the sensitiveness of blue-print paper which was described in this journal in 1891.

The process was originated by Mr. F. H. Latimer, of Vancouver, B. C., and consists essentially in adding oxalic acid to the ordinary blue-print solutions. For the benefit of some of our readers who may make still their own blue-print paper in-

stead of purchasing it, and who may not know of this process for giving it extra sensitiveness so that it will print more rapidly, we reprint the article which was published in our issue of May 23, 1891:

Mr. F. H. Latimer, of Vancouver, B. C., writes us that a year or two ago, while experimenting with blue-printing processes, more particularly with the object of getting bright blues and clear white lines, he found that after the usual washing a bath of quite dilute acid, such as hydrochloric or, better, oxalic, would often greatly improve the clearness of the prints, a marked cause of dirty blues being a gradual altering of the solutions even when kept separate till just before using, though poor quality of the ammonio-citrate of iron seemed to have much to do with the results.

During the experiments he also found that an addition of oxalic acid to the ordinary blue-print mixture materially lessened the time of necessary exposure. The solutions used were:

(1) Ammonio-citrate of iron, 120 grains; water, 1 fluid ounce; to which is added a few drops of strong ammonia solution till the odor is quite percentible.

(2) Potassium ferricyanide, 105 grains; water, 1 fluid ounce.

(3) Saturated solution of oxalic acid.

Equal quantities of (1) and (2) are taken (a); and after being mixed (3) is added as required and the mixture used at once.

Taking, say, in the proportion of 10 ounces of the mixture (a) and adding thereto (b) one ounce, (c) two ounces, or (d) three ounces of (3), the relative rapidity of the coated papers will be closely, in every dull light, as 1:2½:5:10, (d) paper being thus about 10 times as rapid printing as (a) in the light mentioned. For example, a print was made from a tracing on linen in 35 minutes on Feb. 25, 11.30 a. m., on (d) paper during a snow-storm, the light being quite dull, while ordinary paper takes the greater part of a day in an equal light.

This great difference only holds good in dark, cloudy weather; as, if comparisons are made in direct sunlight, (d) paper is only three to four times as rapid as (a). An explanation of this probably is that a weak light which will reduce the oxalic acid mixture (partly ferric oxalate) has but a faint starting or continuous action on the ferric citrate, while with a strong light both commence at once.

For all ordinary purposes it is better not to use a greater percentage than 20% (c) of the oxalic acid solution, as it is difficult to get the lines to wash white with a higher percentage, even with thick black lines on the tracing or negative; and the more sensitive the paper the shorter time it will keep good even in the dark, and also the greater care required in its preparation and use. Some hard close-grained paper that does not absorb much of the iron solution, such as parchment paper (Dietzgen & Co.), Mr. Latimer has found to give the best results in this process, as, indeed, for all blue-print work. To get the clearest prints the paper is better prepared a short time before using in a dark room by gas or lamp light or very weak sunlight.

Ferric oxalate and potassium ferricyanide mixed makes a very rapid printing paper, and can be made considerably cheaper than with the ferric citrate; but it is too sensitive for ordinary work, and requires more care than can be given to it in engineering work, as a rule. Ferric chloride is also more rapid and cheaper, but has the same disadvantages. They seem also to wash off the paper more than the citrate, though different sizing of the paper might remedy this. The only way to get white lines with the two latter, Mr. Latimer has found, is to make the ferric salts frequently, so as to have them fresh, keeping the solutions in the dark or deep red light only.

HYDRAULIC RELIEF VALVES FOR WATER-WHEEL PENSTOCKS.

For the operation of electric generators by hydraulic power, under modern conditions, a precision of speed regulation is required which was never attempted or dreamed of as being necessary in the days when water power was only used to drive the wheels of factories. According to a published statement by a prominent manufacturer of water-wheel governors, a governor must be able to open or close the entire water supply to a turbine in two seconds or less, or it will not be able to effect a speed regulation close enough to meet the demands of the engineers of water power electric plants.

As every engineer familiar with the elementary principles of hydraulics knows, the quicker a valve on a long pipe or penstock is opened or closed, the greater will be the variations of pressure or "water hammer" caused. Thus these modern quick-acting governors have made necessary some provision for taking care of these variations of pressure, not only to protect the penstock from rupture but to prevent this water-hammer action from neutralizing the action of the governor.

As this last idea may not be entirely clear, a word of explanation may be proper. Suppose water is flowing at a certain speed through a long penstock to an exit at one end, through gate-controlled openings, against the vanes of a turbine. Suppose the load on the turbine is reduced one-fourth. Then if the speed is to be held constant, the volume of water delivered to the wheel must be reduced one-fourth; and to effect this the velocity of the water flowing through the penstock must be reduced one-fourth. If the moving fluid were steam or air or other gas, having trifling weight, there would be no difficulty; but water is a very heavier substance, and in order to check the velocity of a long column of water we must in some way absorb its surplus energy. If we do not, the energy will be transmitted to the turbine and the governor will be worse than useless. If this is not evident to any reader, he can settle the question for himself by a very simple experiment. Open an ordinary faucet to its full width, and then close half its opening with the finger. It will be seen at once that the energy of the small stream is much greater than that of the large.

It should be also noted that the energy to be absorbed from even a small reduction in velocity of the water in a long penstock is a very considerable amount. Take for illustration a penstock 4 ft. in diameter and 500 ft. long, through which water is flowing at the rate of 8 ft. per second. The total weight of the column of water will be nearly 200 tons and its total energy will be nearly 200 foot-tons. If now its speed is reduced by one-fourth, its energy will be only $(\frac{3}{4})^2$ of what it was before, or 9-16. Thus, in order to reduce by one-fourth the amount of water delivered by the penstock, we must absorb 7-16 of 200 = 82 foot-tons of work. The difficulty in making any air-cushion device to absorb such an amount of energy is apparent.

For low heads, in a location where trouble from freezing need not be apprehended, a stand-pipe is probably the best pressure regulator for a water-wheel penstock, and it has the great advantage that "it works both ways." That is to say, the water in the stand-pipe will rise and overflow to absorb surplus energy when the gate valve below partially closes; or when the gate valve opens wider to take care of an increased load, the water in the standpipe falls and helps to maintain the pressure while the water in the penstock is acquiring added velocity. But for the high heads which are now so extensively used, standpipes are impracticable and the only available means of controlling the pressure is by some form of automatic water relief valve. It will be apparent from what has been said above that a water relief valve in order to effectively control the pressure in a long penstock must be of very large size, and the difficulty in making a large valve and keeping it closed by direct spring pressure are evident enough.

This problem has recently been attacked by the Lombard Governor Co. of Boston, and a new type of water relief valve has been developed. It is illustrated in the accompanying cuts of which Fig. 1 shows a perspective view and Fig. 2 an elevation, partly in section.

Fig. 1. Perspective View.

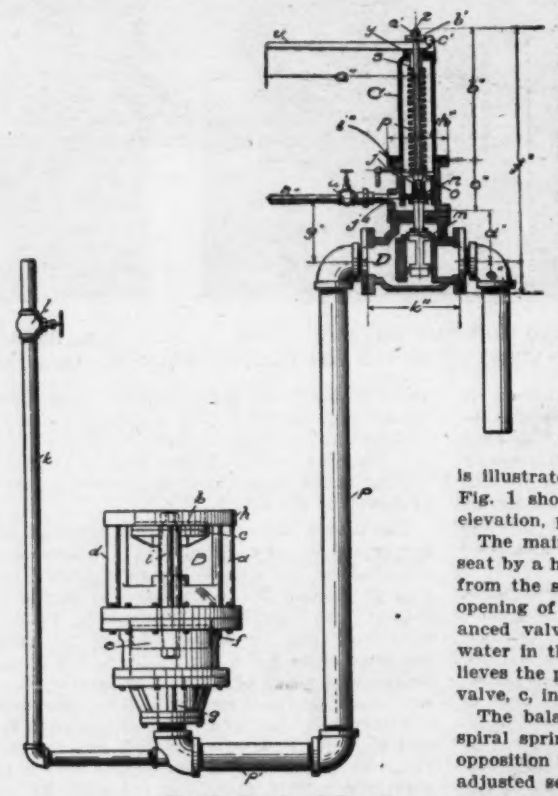


Fig. 2. Sectional Elevation.

AN AUTOMATIC RELIEF VALVE FOR LONG PENSTOCKS AND WATER CONDUITS.
The Lombard Governor Co., Boston, Mass., Makers.

The main relief valve c (Fig. 2) is held to its seat by a hydraulic piston, f, acted upon by water from the supply main through the pipe, k. The opening of the relief valve is effected by the balanced valve D, which, when opened, allows the water in the pipe, p, to run to waste and so relieves the pressure on the piston, f, that the relief valve, c, instantly opens.

The balanced valve D is controlled by a long spiral spring above it and by a piston acting in opposition to the spring. This device is delicately adjusted so that immediately on the water pressure exceeding a predetermined limit the piston, n, rises, opening the valve D and thus the piston f and main relief valve c. To prevent the relief valve from being too suddenly closed, so as to set up the very water hammer it is designed to prevent, or chattering as it goes to its seat, the supply pipe, k, has a throttle valve, l. This can be so adjusted that the relief valve will go to its seat as slowly as desired.

With this arrangement it becomes possible to make a relief valve large enough to do effective work. For example, the largest size listed by the Lombard Co. has a relief valve disk 23½ ins. in diameter, which under a head of 50 ft. should discharge 85 cu. ft. of water per second.

The device would appear to meet a need in the operation of water power plants with long penstocks which has not heretofore been filled.

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Everyone has heard of the person who, for the purpose of escaping from the observation of his fellow men, crawled into a hole and pulled the hole in after him. That mythical feat is now equalled by the achievement of a well-known New York engineer who is driving a tunnel under the Hudson River and actually pushing the hole ahead of him! Speaking seriously, the work on the south tube for the Hudson River tunnel, described on another page of this issue, represents a method of tunnel driving which, we risk little in saying, has never been carried on or attempted before in the world. That it is successful is attested by the rate at which the tunnel is being driven which far exceeds all records of fast tunnel driving ever made.

As described in the article, what is being done is to push the hydraulic shield right into the soft mud, which is forced to either side and upwards into the bed of the river. The tunnel is thus not being "excavated" at all, but it is actually bored in the same manner that an awl bores a hole in wood or leather. Of course, in driving a tunnel in this manner, nearly all the difficulties and dangers usually attendant upon submarine tunneling are eliminated. The workmen are protected by the iron walled tunnel around them and the shield in front. So long as the shield can shove its way ahead without encountering boulders or other obstacle, necessitating the opening of the bulkhead doors and work in front of the shield, the workmen are as safe as if they were in the open air. Moreover, when the shield is pushed forward in this manner, there is actually no tunneling work to be done. All the workmen have to do is to place ring after ring of the cast iron lining as the shield moves ahead. Hydraulic power makes the hole that is ordinarily dug by pick and shovel and dynamite.

It will be evident, of course, that this method of working is only made possible by the exceeding softness of the semi-fluid silt underlying the Hudson River and through which the tunnel is being driven. This semi-fluid silt has defeated every engineer who has hitherto attempted to penetrate it by ordinary tunneling methods. De Witt Haskins spent years of time and a large slice of Trenor W. Park's fortune in driving 2,000 ft. of the north tunnel and the enterprise was then abandoned by both engineers and capitalists. After lying idle for half a dozen years, an English corporation undertook to complete it with the famous firm of S. Pearson & Son as the contractor and the distinguished Sir Benjamin Baker as Consulting Engineer. But the million and a half of money which was raised, proved insufficient to complete even the one river tunnel, and again the work was abandoned and lay untouched for thirteen years.

It was in November, 1874, that work on the Hudson River tunnel was begun, so that 30 years have now elapsed since Mr. Haskins first undertook the work. It has come to be universally regarded by engineers and contractors as the most difficult tunneling work ever attempted. And now comes forward Mr. Chas. M. Jacobs and by simply putting on force enough to push his shield right through the soft silt, discovers that instead of being the most difficult tunneling proposition ever undertaken, it is really the easiest, and he drives his south tunnel along at the astonishing rate of nearly fifty feet per day!

It is doubtless true that the conditions under the Hudson River are extremely unusual. Probably in no other submarine tunnel ever attempted has the material penetrated been soft enough to do with it what is being done now under the Hudson. There is, however, another piece of tunnel work in connection with the same enterprise which represents conditions only too common. The engineer is often asked to drive a tunnel beneath a street surface through soft material loaded overhead by buildings, often of great value. If he judges by the records of past experience, he will probably advise that the probabilities are exceedingly strong that settlement of the building foundations will occur. In driving the Howard St. tunnel in Baltimore, for example, building after building along the line of the tunnel was injured or ruined. In Brooklyn, numerous cases of settlement have occurred along the streets where sewer tunnels have been driven; and other instances might easily be cited.

But the approach to the north tube of the Hudson River tunnel is being built through a coarse, easily moved sand, loaded above by high brick buildings, and the work is being done with such care as to confine the excavated material to the exact section cut out by the shield and so far the buildings above are intact. A more noteworthy achievement in soft ground tunnel driving, it would be hard to find anywhere.

We find in "The Mining World," of Chicago, an editorial entitled "A Case for an Expert," which seems to us so amusing as to deserve a wider publicity. We therefore reprint it in full as follows:

The development and operation of mines, according to modern methods, often present special problems which require special knowledge quite out of the scope of the training of the ordinary mine manager or mining engineer. One such case is told in the following excerpt from an interview given by a leading analytical chemist. He said:

"Not long ago we were retained by the owners of a large Mexican copper mine to discover some alloy that would withstand the attack of a particularly destructive water with which the mine was flooded. Hundreds of thousands of dollars had been expended by the owners for valuable pumping machinery, only to have it eaten up and ruined hopelessly by the action of this water, and they were at their wits' end to discover a remedy. So strong was this water that a steel shovel placed under it would be eaten through and through within [illegible] hours.

"A hogshead of the water was sent to us in New York for analysis, and we found it chockfull of sulphuric acid. The problem was, therefore, to find some alloy that would resist the action permanently. The experiment was absurdly simple. We sent for samples of 400 different bars of alloys. Each little bar was carefully cleaned and dried and weighed and immersed in a small jar containg exactly half a pint of the water submitted. At the end of a week, each bar was removed and dried and weighed on a hair balance. Out of the 400 only 12 showed they had suffered no loss in weight.

"With these 12 we experimented as before, until at the end of a month we found four of the bars would answer the purpose, not having lost the least perceptible weight throughout the test. Of the four we advised the cheapest, which, it happened, was also the most durable as far as wear and tear is concerned, and from that day to this there has been no trouble at that mine, excepting that the water eats the shoe nails out of the boots of the miners."

While the analytical chemist can take credit for a clever piece of work, credit must also be given to the intelligent mine manager, who knew when he had encountered a problem outside of his knowledge and experience, and who had the wisdom to call in the services of a scientific expert to give him the information required.

For our part, we are consumed with curiosity to know what the "four alloys" can be which will withstand without corrosion exposure for a month to a sulphuric acid solution strong enough to eat through a steel shovel blade in 24 hours! We can pardon the "leading analytical chemist" for his modesty in withholding his name from publication; but for the benefit of the world at large he ought to let it be known what these remarkable alloys are which by his own account are as resistant to corrosion as gold or platinum, and yet are cheap enough to use for lining a mine pump. Then, too, his laboratory processes are so unusual! They may seem "absurdly simple" to him; but many of them will certainly puzzle his brother chemists. Why, for example, did he try 400 alloys haphazard, when any standard treatise on chemistry records that of all the metals, only gold, platinum, iridium and rhodium are unattacked by sulphuric acid. Why did he take "exactly half a pint" of the water in which to immerse the test bars; and why, oh why, did he not advise the owners to work their mine for sulphuric acid instead of for copper! Had he done this latter, we might then have joined with our contemporary in crediting him with "a clever piece of work."

Conditions like the following are so frequently found in specifications that a few words of disapproval will scarcely have any effect, no matter how well founded the objection may be.

Bidders shall specify in their bids the number of working days they will require for the completion of the work.

The principal objection to this form of bidding lies in the unknown value that will be set upon each day. Will the awarding official count each day as being worth $5, or $50, or $150? Unless it is known beforehand what value will be set upon the unit of time, the day, a contractor is at a loss how to bid. Moreover, he is apt to infer that favoritism will govern the letting, even if no favoritism is intended; for it is evident that a favored contractor can unbalance his bid simply by bidding high unit prices and naming a very short time limit. To illustrate what an effect this uncertainty has, let us suppose that A bids $50,000 for the work, naming 200 days as the time of completion, and suppose that B bids $54,500, naming 100 days as the time required. Obviously A is entitled to the contract if the value of a day's time is assumed to be $25, for we have:

A's bid	$50,000	B's bid	$54,500
200 days, at $25	5,000	100 days, at $25	2,500
Total	$55,000	Total	$57,000

But suppose the awarding official decides that $50 is the worth of each day's time, then conditions are reversed, for we have:

A's bid	$50,000	B's bid	$54,500
200 days, at $50	10,000	100 days at $50	5,000
Total	$60,000	Total	$59,500

No public works letting should ever leave it in the power of an awarding official thus to juggle with the bids received. To avoid the appearance of unfairness, if for no other reason, such clauses as the one above quoted should be excluded, unless a definite value is named for each day's time to be used in comparing bids.

The same specification from which the foregoing clause was abstracted contains the following:

The Commission reserves the right to increase or decrease the work, or to stop the work, wholly or in part, at any time. Nor will the Commission be liable for any loss or damage to the contractor because of any stoppage of the work.

A number of complaints from contractors contemplating bidding upon this work seem to justify giving space to this clause, in spite of the fact that similar clauses have been previously discussed in the columns of this journal. It appears not to have occurred to the author of this specification that the element of time is a factor of importance to a contractor, or that the amount of

work to be done affects the unit cost, or that there is any unfairness in a specification that permits one party of a contract to escape from the implied obligations while rigidly holding the other party to performance. Nor does it seem to have been regarded as possible that some contractors might hesitate to buy a plant or part of a plant to do work which may be shut down before the plant has been in operation long enough to pay the freight from the factory. Every clause like this brings its own punishment either by causing only a reckless class of contractors to bid or by causing conservative contractors to bid an exceedingly high price to cover risks. There are internal evidences of lawyer authorship of these specifications; and it is not improbable that if an engineer ever had a hand in the original draft, he has been forced to see all semblance of fairness wiped out by a few legal—but foolish—pen strokes; for it is an economic folly to attempt to play fast and loose with intelligent and reliable contracting firms.

THE DIFFERENCE BETWEEN HARDPAN AND ROCK.

Hardpan is a material that may be regarded geologically as being rock in the process of formation. Any clay that has become so hardened by heat or pressure as to be an incipient shale, is hardpan. Any sand that has been partly cemented by the deposition of a small amount of iron oxide or carbonate of lime in its pores, is also hardpan. To one who is ignorant of geology, the word "rock" conveys the idea of an exceedingly hard and practically solid mass; but every student of geology knows that in fact there is no marked line dividing rock from earth, the one passing insensibly into the other. This follows naturally from the fact that sedimentary rocks are merely hardened sediments; sandstone, for example, is cemented sand and shale is baked clay. Obviously the processes of solidification, be they physical or chemical, may be found illustrated in nature's laboratory in all stages from the softest clay, through hardpan and shale, to the hardest slate. "Obviously," we say, yet notwithstanding the obviousness, even among engineers who should know better, there are many who still use the word rock as if it applied only to some definite kind of the earth's crust.

Although the subject of rock and earth classification has been discussed in our columns before, attention is again called to the matter by the prominence given in the daily papers of New York state to this subject. Without entering into the political reasons actuating the daily papers in again bringing before the public an old discussion, we shall quote a statement made by one of the political orators regarding some excavation on the Erie Canal, which several years ago was classified as hardpan. Here is the orator's statement:

> I am no more a geologist than I am a farmer, but I know enough about farming to know that corn cannot be grown on rock, and I know enough about geology to know that the stuff which we scooped up in our hands from that pile is not rock. In one place the deposits of this fraudulent "rock" had great holes scooped out of them by farmers who had hauled the stuff away for a top dressing for their fields because the earth is filled with fine shells which have a most beneficial effect on the soil.

This orator who admits that he is no geologist —nor is he an engineer—is sure, however, that material classified as rock was fraudulently so classified by the engineers. It should be stated that the specification under which the classifications of this Erie Canal excavation were made, prescribed the plow test to mark the dividing line between earth and rock, as will appear from the following quotation:

> Dry excavation of rock will include the removal of all solid or ledge rock and *hardpan* which, in the opinion of the resident engineer, cannot be plowed, and all boulders measuring one-half a cubic yard or more.

We have italicized the word hardpan to indicate that the authors of the specification meant to include this incipient form of rock if its hardness were such as to prevent plowing. What the public can not understand is how a material which is so hard that it can not be plowed, becomes, after five years of exposure to heat and cold and frost, a disintegrated mass which farmers "scooped out," and "hauled away for top dressing."

The fact is that many granites, which are among the toughest of rocks, disintegrate in the course of time when exposed to the climatic conditions of New York state. The Egyptian obelisk in Central Park is rapidly crumbling away; and hundreds of brownstone (sandstone) fronts in New York city have long ago fallen into a dust that would make good farming soil—indeed, it is in just this way that all soils are made from the rocks. What surprises any one but a geologist is the rapidity with which certain rocks disintegrate If the disintegration of building stones is rapid, even more rapid is the crumbling of the softer stones that are unfit for building, and still more rapid is the destruction of the incipient stone commonly called hardpan.

To illustrate that this fact is well known to many engineers, we quote from a paper on the Chicago Drainage Canal by Mr. E. R. Shnable, which was published in the "Journal of the Association of Engineering Societies" for June, 1895:

> This hardpan does not soften in the least under water when first taken out of the pit. After a piece has been exposed to the air until dry it dissolves rapidly upon immersion.

By the word "dissolves," Mr. Shnable does not mean dissolving like sugar in water, but crumbling into minute fragments—a phenomenon well known to all who have had experience in clay hardpan excavation. It is indeed surprising, but it is nevertheless a fact that a hardpan which can not be plowed on account of its toughness, nor readily blasted on account of its lack of brittleness, will crumble to pieces upon exposure for a few months, particularly in the winter and spring.

Our purpose is not to defend the plow test, but to impress upon engineers in charge of public works the danger that is run wherever this test is prescribed, because of the ignorance with regard to rock formation that prevails outside of scientific circles and even within them.

The fact is that no specification prescribing a physical test to mark the line between earth and rock, has thus far proved satisfactory in the long run. This truth is recognized by the engineers of many railway companies who specify no classification at all, preferring to avoid lawsuits by lumping both earth and rock together under the one head, "excavation"—a procedure that is even less defensible than the plow test, since it leads to guesswork or gambling on the part of the contractor, which in the end means an unreasonably high cost for excavation. Moreover, it reacts upon the engineers themselves, making them careless in their study of subsurface conditions, and careless in their study of the unit costs of excavation.

On public work involving excavations of magnitude, the specifications should be drawn to cover each class of material as determined by actual borings and test pits, samples of the materials being labeled and filed for reference in interpreting the specifications. There is no other safe method of classifying excavation that ranges through all grades of toughness and hardness.

LETTERS TO THE EDITOR.

A Plea for Small-Sized Contracts on the New York Barge Canal.

Sir: Is the letting of contracts on the Erie Canal to be free from scandal? That remains with the policy to be pursued by the State. If the same line is pursued as was done on the $9,000,000 enlargement, scandal will be sure to follow.

The State went on the line of having large cash deposits with each bid and continuing them through the existence of the contract, and also of exacting large bonds for the execution of them. The consequence of such a policy is that a monopoly is made of the work, as comparatively few people are able to comply with such requirements.

The bidders being few, it is not difficult to learn who they are; and, as it is reported that the work is to be let in amounts of about $1,000,000, they put their heads together and divide the work among themselves at prices to suit. This makes it impossible for any man of good mechanical and executive ability to be a bidder unless he has great financial responsibility, as the surety companies would not furnish him a bond.

The only people that they will furnish bonds are of undoubted financial responsibility.

No bonds should be required except from the engineers and executive officers in charge. The state can pay every two weeks for work done. If the work is not done or poorly done—no pay.

This would make the officers look out for men of good moral character and mechanical ability who would have a pride in doing their work well.

As the contractors that take the work in large amounts will sublet in smaller amounts, why is it not the business of the State's officer to do this instead?

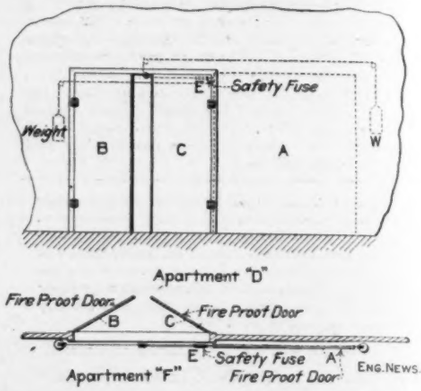
I am certain that if the bars are thrown down the work will be done 20% less than otherwise, and it will bring to the front men who are experts in their line and will take the work with the expectation of making something out of it on account of this expert knowledge and not through any monopoly of bonds and large cash deposits.

Yours,
Anti-Bond Monopoly

Buffalo, N. Y., Oct. 16, 1904.

A Case of Too Much Protection in Fire Resisting Construction.

Sir: In the pattern storehouse of one of the large Rhode Island machinery concerns, there is a case of inspection which savors of over-requirement and obstinacy. The accompanying sketch represents a door in the three-story pattern house which is used entirely for storing wooden models. This building, which is lighted by electricity taken from the city mains, is always kept locked and is heated neither in cold nor in warm weather. Each floor is divided into two parts connected by a doorway upon which operate the fireproof swinging doors B and C and the fireproof safety sliding door A. In case a fire breaks out in apartment F, the heat melts the fusible link



at E, and breaks the connection, causing the door to shut and protect room D. But should a fire break out in side D and door C should happen to be partially or entirely closed, neither the heat nor the flames would attack the fusible link to make door A shut for the protection of the adjoining apartment. Although this possibility was pointed out to the inspector by the general manager of the concern, the former insisted on both sets of doors remaining where they were instead of having the swinging ones removed. This case is merely an incident amounting to practically nothing in the way of financial loss to the concern in question, as there is almost no possibility of a fire starting in the interior of the structure. At the same time, it would be well to have inspectors use a little more common sense and reason instead of insisting on the letter instead of the spirit of protection requirements.

W. T. M.

Providence, R. I., Oct. 22, 1904.

Wind Stresses in Knee-Braced Mill Buildings.

Sir: The writer would like to discuss somewhat the communication by Mr. W. H. Dunham, on "Wind Stresses in Knee-Braced Mill Buildings," in Engineering News of Oct. 6, 1904.

As a usual thing this matter is not very carefully considered. It seems to be the usual practice to assume a loading of x lbs. per sq. ft. of horizontal projection supposed to be sufficient to take care of wind strains. This is usually sufficient, so far as the chords are concerned, but where knees are needed and used, stresses are developed in some of the web members that this method will not provide for, and especially so if the building be high in clerestory. The correct computation of the stresses in a truss under such conditions requires too much work and is unnecessary in the majority of cases.

The writer has seen numerous buildings in which little or no attention was paid to the wind stress in the columns, especially in their anchorage and their connection to the truss. Again, he has seen cases where knee-braces were provided, perhaps strong enough themselves, but which were insufficiently connected, and where the web members of the trusses, which have to carry most of the stresses, were not proportioned for those stresses. Why do such buildings stand up? Doubtless because they have a large nominal factor of safety and because they never get the wind load which they are supposed to

carry. The logical conclusion would be, if the latter be the case, to use lighter wind loads and provide for them.

But to come to our present subject: it seems to the writer that Mr. Dunham in his criticism of Mr. Hutchinson has presented a rather erroneous proposition. Taking his stress diagram and following it from left to right till we come to the point where the members NG, GI', I'J' and J'N meet, we will find that for this point his stress diagram does not close by the amount of the vertical component of the stress in a knee brace. This alone is sufficient to spoil the proposition; but further we cannot make the diagram close until we correct his reactions by taking moments of the horizontal wind force (as applied at the bottom chord) about the foot of the knee brace. This will increase the leeward reaction by 4,440 lbs. Then the diagram will close.

But even when so corrected, the diagram, while closing, does not give the true stresses; for, it is made on the supposition that the column can carry no bending at the foot of the knee. Therefore, we are forced to two conclusions: First, Mr. Dunham's diagram is erroneously drawn for the loading he assumes; second, were it drawn correctly, there could be no moment in the column as he deduces.

Again, from Mr. Dunham's analysis, it would not matter how high the columns were, so long as the total wind pressure were the same; or going further, by attaching the knee only 6 ft. down instead of 12 ft., he would require only half as strong a column.

Now to try to get at the correct solution: so far as the columns are concerned, we note that the truss ends at the foot of the knee braces. From that point the columns must carry the wind load as a constant shear to the foundation. This shear produces bending in the columns. The most favorable condition for the column and also for the knee-brace, is with the columns fixed at the base. This condition can usually be obtained in practice.

Then for a column of uniform section, considered fixed at the bottom, the bending moment equals the shear times half the height to knee brace; or, for the case under consideration, $11,850 \times \frac{1}{2} \times 30.5 = 180,712$ ft.-lbs., instead of 142,200. Then for the horizontal component of the knee-brace, we have $11,850 \times (\frac{1}{2} \times 30.5 + 12) \div 12 = 26,910$ lbs., a far different stress from that given by Mr. Dunham.

Moreover, from this we see that if the knee brace were attached 6 ft. from bottom of truss, the stress would be greatly increased, as we would naturally expect.

As to the stresses in the truss, we have in addition to the reaction at the foot of the knee braces, a moment as given above to deal with. It is not the writer's purpose here to extensively discuss the stresses produced, but by inspection we note that this moment becomes zero at the center of the symmetrical truss in question, and becomes only a shearing force, which into the lever arm of half the span of truss equals the given moment. This enables us to get all the stresses in the truss members due to the bending, if we desire.

The writer usually figures the truss for a uniform load as noted in the first of this article, and then finds the effect of the knee stress on those web-members most seriously affected, increasing their section if necessary. Of course, that is not thoroughly scientific, but it is quick, and safe enough.

Yours truly,
Chas. M. Emmons.

Beaver Falls, Pa., October 13, 1904.

Sir: The writer has been very much interested in the proposed solution for the stresses in a transverse given in your issue of Oct. 6, 1904. The problem of calculating the stresses due to wind in a transverse bent is quite a complicated one and quite a number of solutions have been proposed. Many of these solutions differ considerably on account of the different assumptions as to the condition and behavior of the columns and the knee-braces; while other solutions, among which is the one proposed by Mr. Dunham, are based on assumptions which are certainly open to question.

The solution proposed by Mr. Dunham is open to the following criticisms:

(1) The point of contraflexure is certainly not located at one-half the height of the column as assumed; if the column is fixed from the knee-brace to the top of the column, the point of contraflexure will be midway between the knee-brace and the base of the column, while if the foot of the knee-brace and the top of the column deflect an equal amount, the point of contraflexure will be (in this particular case) 17 ft. above the base of the column. Hutchinson assumes the point of contraflexure as 13 ft. 6 ins., while Dunham assumes that it is 21 ft. 3 ins. from the base of the column. Both results are certainly erroneous.

(2) The horizontal components of the wind loads at the panel points in the top chord appear to have been omitted in calculating the stresses in the truss.

(3) On account of the manner in which mill buildings are constructed, part of the wind should be considered as concentrated at the foot of the knee-brace. This would make the stress in the windward knee-brace less than the stress in the leeward knee-brace. Dunham gets the same stress in both knee-braces.

(4) The maximum bending moment in the column as assumed by Dunham will come at the base of the column and not at the foot of the knee-brace. A calculation of the pedestals required to fix the column will show that all that might be gained by his solution in the trusses will be more than counterbalanced by excessive cost of foundations. In fact, in many cases with stresses properly calculated it will be found economical to calculate the stresses and design the bent for columns hinged at the base, rather than go to the expense of building pedestals sufficient to fix the columns.

It would appear from the above that the solution proposed is neither theoretical in its assumptions, nor practical in its application.

In the writer's book on "Steel Mill Buildings"* he has developed a solution for calculating the wind stresses in a transverse bent which is simple and direct. The solution is illustrated by the accompanying cut, which is taken from Fig. 57 of that work.

In developing this solution the writer shows (1) that the point of contraflexure for ordinary cases varies from one-half to five-eighths of the distance from the base of the column to the foot of the knee-brace, and that we will be on the safe side if we take the point of contraflexure at one-half the distance from the base of the column to the foot of the knee-brace; and (2) that as far as the stresses in the bent are concerned, the columns in a bent with columns fixed at the base may be considered as hinged at the point of contraflexure, the wind above that point only being considered.

To remove the bending in the columns, auxiliary members are substituted as shown, and the problem is then solved by simple statics. By the substitution of the auxiliary members the bent becomes a two-hinged arch with the horizontal reactions known. It will be noted that the wind load below the point of contraflexure is omitted. The stresses in all members of the truss between the columns are correctly given by the stress diagram. The stresses in the columns are easily found algebraically.

The writer's solution of the bent shown in your issue of Oct. 6, with the point of contraflexure assumed as 17 ft. above the base, gives — 18,300 lbs. and + 45,000 lbs. stress in the windward and leeward knee-braces, respectively; as compared with — 18,600 and + 18,000 by Dunham, and — 32,300 and + 54,800 by Hutchinson for the corresponding members.

In designing columns for wind moment the writer's specifications allow a working stress of 20,000 lbs. per sq. in. for the combined stresses.

Very truly, M. S. Ketchum,
Professor of Civil Engineering, University of Colorado.
Boulder, Colo., Oct. 20, 1904.

*"The Design of Steel Mill Buildings and the Calculation of Stresses in Framed Structures"; Engineering News Publishing Company, New York City.

Sir: In your issue of Oct. 6, 1904, Mr. Dunham makes the following remarkable statement:

The action of knee-braces in resisting the wind forces on the sides and roofs of mill buildings, and their effect on the roof trusses and columns is something of an open question among structural engineers, and is not usually given the attention it deserves in the design. Most writers either ignore the subject entirely or treat it in a superficial manner, while the few who have really attempted its solution obtain results so at variance with accepted practice as to render their results of no practical value.

I think this subject is quite well understood by all competent engineers, and that it receives from them all the attention it deserves. Furthermore, it is fully analyzed in several works of high standing, such as: Greene's "Graphics," Johnson's "Framed Structures," and Ketchum's "Mill-Building Construction." In all of these books practically the same method is employed.

Mr. Dunham's method is not very clearly explained, but, since he has taken the horizontal reactions at the foot of knee-braces to be equal to the total horizontal force on roof and at top of post, he evidently assumes that the frame would turn about the foot of knee-braces. Under this assumption there would be no bending moment in posts at knee-brace connections, which is absurd.

If Mr. Dunham's method were in general use I think there would be many failures in structures of this kind.

The following is the generally accepted method of solving the problem, and the results obtained thereby are probably very near the actual conditions.

The building used by Mr. Dunham is taken as an example. The dimensions of this and the loads are as follows:

Width, center to center of posts, 64 ft.; trusses 32 ft. apart.
Dead load, 25 lbs. per sq. ft. horizontal projection.
Wind load, 20 lbs. per sq. ft. vertical projection.
Dead load per panel = 32 ft. × 8 ft. × 25 lbs. per sq. ft. = 6,400 lbs.
Wind on side of building = 32 × 42.5 × 20 = 27,200 lbs.
Wind on side of monitor = 32 × 7.5 × 20 = 4,800 lbs.
Wind at panel points of truss, horizontal component = 1,500 lbs.
Wind at panel points of truss, vertical component = 3,000 lbs.

One-half of the wind force on side of building is applied at foot of post and may be neglected for the present as it has no overturning effect on building. The other half, 13,600 lbs., is applied at top of post. One-half of the wind force on side of monitor is applied at top of monitor post, and one-half at the foot.

The total horizontal force to be considered is as follows:

	lbs.
At top of post	13,600
On main roof 1,500 × 3 =	4,500
On side of monitor	4,800
On roof of monitor	1,500
	24,400

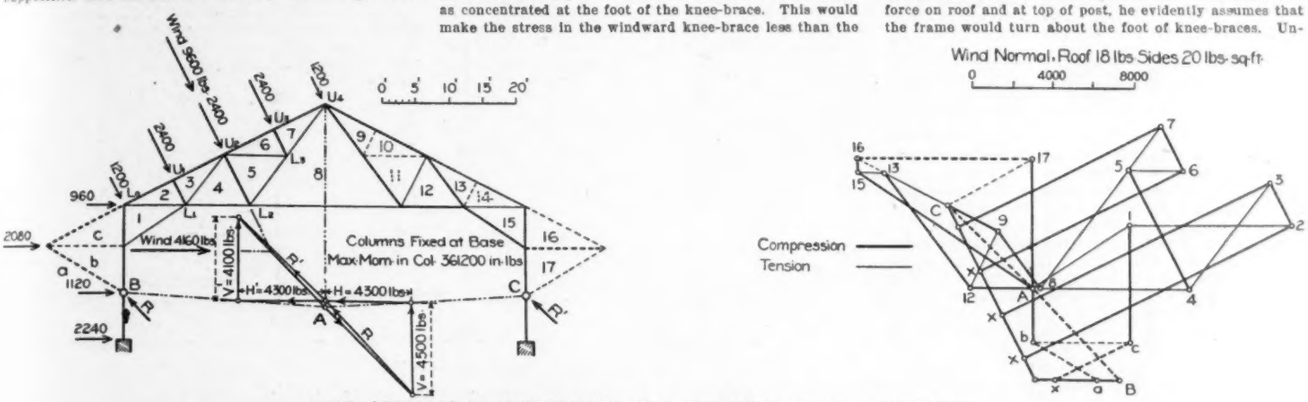
one-half of which, 12,200 lbs. is assumed to be resisted at the foot of each post.

If the posts were free to rotate at their base the vertical reactions due to the horizontal forces would be obtained by taking moments of these forces about the foot of posts, and dividing by their distance centre to centre. But the posts are more or less fixed by the dead load of roof and walls, also by the anchor bolts if properly built into the foundations. Consequently there will be a point of no moment somewhere between base of post and foot of knee brace. This point of no moment, or of contraflexure, should never be assumed higher than half way between base of post and knee-brace connection. The existence of a resisting moment at the foot of each column changes the vertical reactions from those determined by pure statics.

Taking the weight of roof at 25 lbs. per sq. ft. and of side covering at 20 lbs. per sq. ft. the load on post is as follows:

Roof = 32 ft. × 32 ft. × 25 lbs. = 25,600 lbs.
Side = 32 ft. × 42.5 ft. × 20 lbs. = 27,200 lbs.
52,800 lbs.

This force acts at center of post. Then, assuming the width of base to be 2 ft., the moment of resistance of the base is 52,800 lbs. × 1 ft. = 52,800 ft.-lbs. The anchor



WIND STRESSES IN KNEE-BRACED MILL BUILDINGS.—BY M. S. KETCHUM.

This cut is a reproduction of Fig. 57 in "Steel Mill Buildings."

bolts will not be considered in the present case. The horizontal reaction at foot of post, multiplied by the height to point of contraflexure is equal to the moment of resistance of base; therefore, height to point of contraflexure $= \frac{52,800}{12,200} = 4.32$, say 4.5 ft. With efficient anchor bolts this point might be twice as high, or even higher.

For the vertical reactions due to horizontal forces, moments of these forces are taken about the assumed plane of contraflexure.

Vertical Reaction N A

14,350	× 38	=	546,000
1,500	× 42	=	63,000
1,500	× 46	=	69,000
3,150	× 50	=	157,500
3,150	× 57.5	=	182,000
750	× 61.5	=	46,000

Horizontal wind.....1,063,500 ft.-lbs. ÷ 64 = — 16,600
Dead load = 6,400 × 4..... = + 25,600
Vertical wind = 3,000 × 4 × ¾..... = + 9,000
+ 18,000

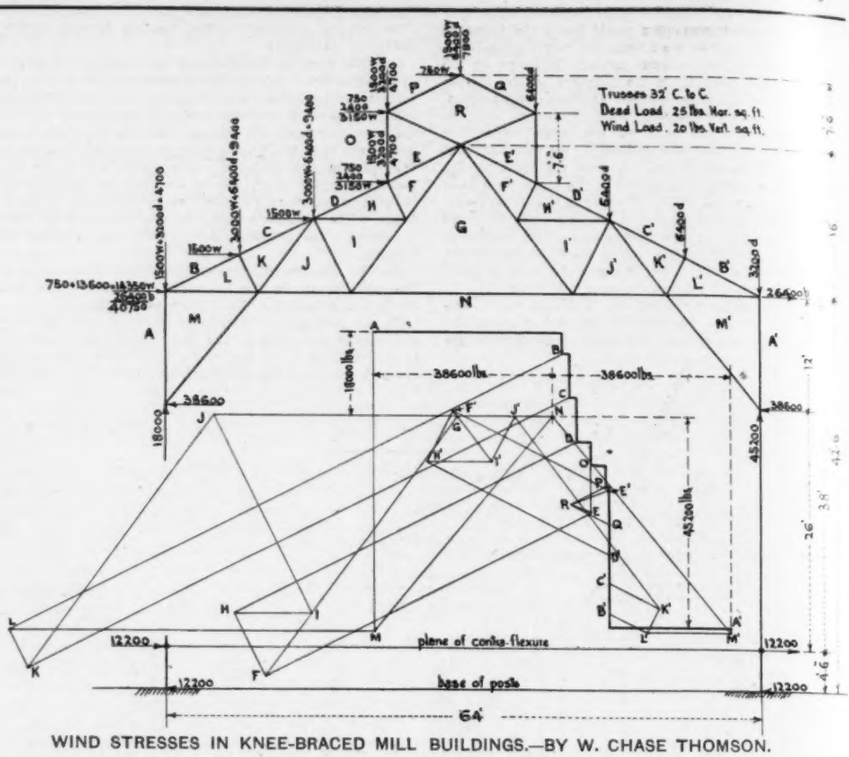
Vertical Reaction A' N.

Horizontal wind..... = + 16,600
Vertical wind = 3,000 × 4 × ¾..... = + 3,000
Dead load = 6,400 × 4..... = + 25,600
+ 45,200

The maximum moment in posts is at foot of knee braces and is equal to 12,200 lbs. × 26 ft. = 317,000 ft.-lbs.

Horizontal shear at top of post due to this bending moment = 317,000 ÷ 12 = 26,400 lbs. Horizontal force at foot of knee-brace = 12,200 + 26,400 = 38,600 lbs. The horizontal and vertical loads are now laid off in the stress diagram in regular order from A to A', and are shown in heavy lines; also the reactions A' N and N A. The stress diagram is then proceeded with in the usual way, beginning at the lower end of either knee brace. The stresses obtained from this diagram are as follows, in which + = compression, and — = tension.

B	L	+ 133,000
C	K	+ 130,000
D	H	+ 83,000
E	F	+ 79,000
F	G	— 70,000
F	H	+ 15,000
G	I	— 53,000
H	I	— 17,000
I	J	+ 47,000
J	K	— 68,000
K	L	+ 9,000
L	M	— 78,000
J	N	— 73,000
G	N	— 20,000
M	N	— 60,000
B'	L'	— 9,000
C'	K'	— 12,000
D'	H'	+ 44,000
E'	F'	+ 38,000
F'	G	— 2,000
F'	H'	+ 12,000
G	I'	+ 12,000
I'	J'	— 11,000
J'	K'	+ 52,000
K'	L'	+ 6,000
L'	M'	— 18,000
J'	N	— 8,000
M'	N	+ 60,000



WIND STRESSES IN KNEE-BRACED MILL BUILDINGS.—BY W. CHASE THOMSON.

Assume a girder section of

1 — 24 × ⅜ pl. = 9.00
4 — 6 × 3½ × ½ = 18.00
27.00 sq. in.

Section modulus = 209.
Then 64,000 ÷ 27 = 2,400
3,804,000 ÷ 209 = 18,200
Maximum compression outer fibers..... = 20,000 lbs. per sq. in.

This unit stress is quite safe, as the posts will be held sidewise by the wall framing.

Yours truly,
W. Chase Thomson,
Assistant Engineer, Dominion Bridge Co., Ltd.
Montreal, P. Q., Oct. 14, 1904.

Sir: Referring to the letter in your issue of Oct. 6th, by Mr. W. H. Dunham, on "Wind Stresses in Knee-Braced Mill Buildings," it may be of interest to present a solution of this problem in accord with a theory developed by me and elaborated in a paper published in Transactions Am. Soc. C. E., Vol. XLVIII, 1902. I have used this method in designing columns of the type under consideration in many cases during the last few years and believe it combines legitimate assumptions, simplicity and flexibility of application, and final results that appear rational.

The assumption that columns of this type are "fixed" at the base in the sense ordinarily used in the subsequent application of the theory, is not proper, in my opinion, for there would be required a base having the full section of the column or its equivalent in a splice, to fulfill the requirements of this assumption, and even then, supposing the column section itself were to extend down into the foundation and all play between it and the masonry eliminated, the critical point would only be transferred lower down to the joint between the masonry and the ground on which it rests.

Applying now my theory to the problem presented in Mr. Dunham's letter, I have shown in Fig. 1 the forces acting on the structure as a whole, the reactions being determined by the ordinary principles of statics; assuming also that the horizontal thrust of the wind is divided equally between the windward and leeward columns.

In Fig. 2 is shown, exaggerated, the effect of a horizontal thrust on the frame. The columns tend to bend to the shapes indicated, and in so doing the windward edge of the column base tends to rise, throwing all the vertical

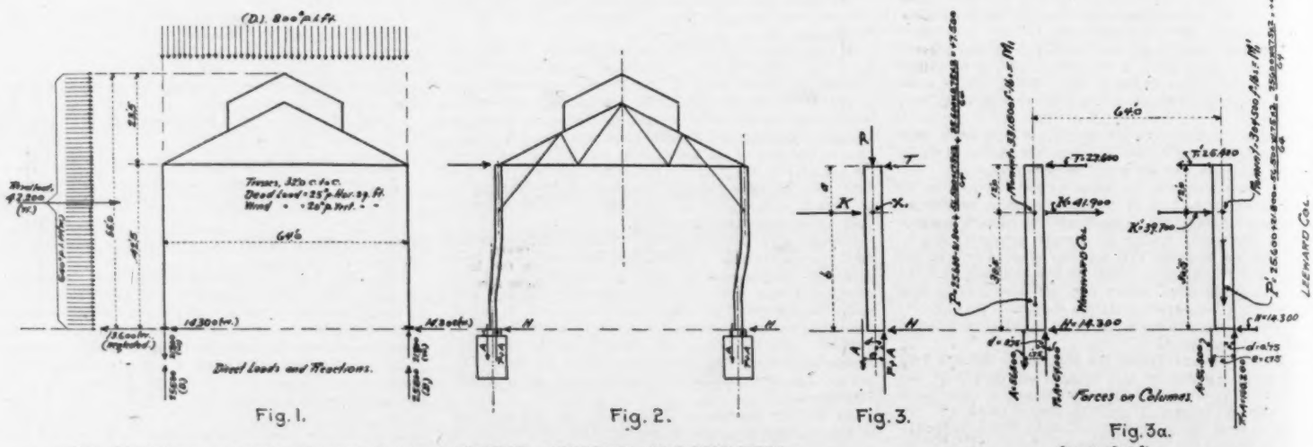


Fig. 1. Fig. 2. Fig. 3. Fig. 3a.

WIND STRESSES IN KNEE-BRACED MILL BUILDINGS.—CHARLES WORTHINGTON.

If the plane of contraflexure had been assumed higher the stresses would have been correspondingly less, but this would not affect the method.

It is usually more convenient to make separate stress diagrams for the wind force and for the dead and snow loads.

The posts should be designed for the combination of direct compression and bending moment.

Compression in A M from diagram = 64,000 lbs.

Bending at foot of knee = 317,000 ft.-lbs. = 3,804,000 in. lbs.

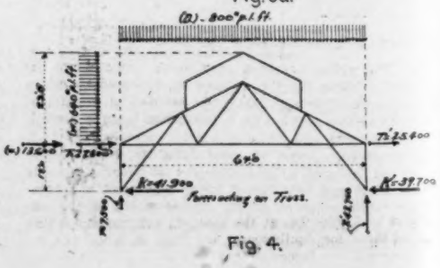


Fig. 4.

load upon the leeward edge of base, and bringing the windward anchor bolts into play; the forces and their points of application being as indicated. The amount of distortion in actual columns would be very slight, so the condition of the column-loading may be assumed as in Fig. 3. The frame will now be affected by two additional conditions, namely, the moment produced by the anchor bolts, of total amount 2 A e for the two columns (where A is the stress in anchor bolts, and e the distance between the windward anchor bolts and edge of column foot, as indicated in Fig. 3); also the moment produced by transferring the point of application of the dead load from the center to one edge of the column foot, of amount equal to the product of the vertical load in each column by the horizontal distance d (see Fig. 3) between the center of column and edge of column foot. These two moments act in a direction opposite to the wind moment, and divided by the distance between the columns give reactions, of equal amount and opposite direction, in the windward and leeward columns in addition to those shown in Fig. 1. In Fig. 3a are shown the resulting forces acting on the columns of the building under consideration; this drawing is in most part self-explanatory. The force A represents the stress in anchor bolts and is assumed. In the present case I have used four 1½-in. diameter (upset) bolts to each column, two on the windward and two on the leeward side; assuming further a unit stress of 16,000 lbs. makes two of the bolts, or the value of A, equal to 56,500 lbs. as indicated. The columns are assumed to be of same width, 18 ins., as those used in Mr. Dunham's design.

The forces P and P′ are derived by combining the reactions indicated in Fig. 1, with those resulting from the introduction of the forces A and from shifting the dead load reaction to the edge of columns. The critical moment in the column will be at X_1 (see Fig. 3), the intersection of knee-brace and center of column, and may be written in general thus,

$$M = Hb - Pd - Ae \quad \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

Examining equation (1) it will be noted that both the anchorage A and the vertical load P tend to reduce the bending moment in the column, as does also increasing the width of the column or the distance between anchor bolts. In tall buildings where the direct load P is large, especially in the lower stories, the bending is of small importance compared with the direct load.

The column foundations must be designed for the loads as shown in Fig. 3, care being taken to develop the full value of the assumed pull in the anchor bolts. The bottom end of the column must be designed to bear the total load (P + A) on one edge without crushing; this is usually provided for in the ordinary details of the base.

Returning now to Fig. 3a, the moments M_1 and M_1' may next be written from equation (1) above, and dividing same by the distance a (= 12 ft.) will give the forces T and T′, respectively; K and K′ next follow as the sums T + H and T′ + H′, respectively.

We now have all the stresses in the columns from which their sections may be designed. The assumed width of 18 ins. appears to be too small for economy in these columns.

In Fig. 4 are shown the resultant forces which will act upon the truss and for which the stresses in the members are easily determinable.

A much cheaper and equally effective method of providing for the horizontal thrust produced by wind, traveling cranes, etc., on buildings of this type, is to transfer same to each end of building by means of bracing in plane of bottom chords of the trusses, then directly down the ends of building by diagonal bracing. Where this method can be employed wholly or partially it will be found to use much less material than performing the same service by bending the columns.

Referring now to Mr. Dunham's design, it appears to be unnecessary refinement to assume that the wind acts normal to the roof surface; the assumption that the wind acts horizontally upon the vertical projection of exposed surfaces is, in my opinion, as close to the actual conditions as one can get and is safe, especially when it is considered that under a strong wind, which develops gradually, the snow will be evaporated or blown from the windward side of the roof. Such hair-splitting refinement in assuming the loads on an ordinary roof when there is so much uncertainty in the actual distribution of the loads, by reason of the well known fickleness of the wind, variation in the amount and location of the snow loading, the roughness of construction ordinarily employed in this class of work, etc., only adds to the labor of calculation and increases the chances of error without producing results that will vary materially from those obtained by less refinement in assumption of loading. It does not seem right to divide up the wind load so carefully and then proceed to make calculations on the loose assumption that windward and leeward columns will each take just half the total load.

In Fig. 2 of Mr. Dunham's letter the horizontal force acting on the column at the knee-brace connection and that at the top are shown equal; this appears to be incorrect, for while the diagrams do not show the horizontal reactions of 11,850 lbs. at the foot of each column, they must be there for equilibrium, and then equality cannot exist between the forces at top and at knee-brace connection as shown, but the latter must be greater than the former by the amount of this reaction of 11,850 lbs.

It would be interesting to examine a completed diagram of loads and reactions embodying Mr. Dunham's assumptions. He appears to have assumed, for designing the truss, that the columns are broken at the knee-brace connection and the truss is just resting there; this is the only condition under which the horizontal force at top of column and the reaction at knee-brace connection will be equal. But this condition would require the column anchorage to resist a moment of 11,850 × 30.5 = 361,000 ft.-lbs., instead of the 4,000 ft.-lbs. which he gives, apparently, for the anchorage at base of column. On the other hand, Mr. Dunham has given a bending moment for the column at point where the knee-brace connects, which would seem to indicate that he does not assume the column to be broken here.

It appears upon examining his Fig. 2 that the lower half of the wind on side of building is resisted by the windward column acting as a cantilever. Why not the upper half as well? As a matter of fact the windward column resists the wind on this side as a beam of length 42.5 between its end supports, independent of its main function as a column. This beam effect is generally neglected as of small consequence, and the load on side of building assumed to be transferred by this beam to its ends, one-half to the top, and the other to the bottom; the latter portion need not be considered further, as it does not affect the structure excepting possibly by tending to shear off the anchor bolts of the windward column.

Another question I would like to bring up is how in Mr. Dunham's Fig. 3 the windward reaction is shown greater than the leeward, when as a matter of fact the windward, which, for the horizontal component of the wind forces alone, would be equal in amount to the leeward, is reduced somewhat by his assumption of a vertical component of the wind acting on the windward side only, and still further by the dead load; while the leeward will be increased considerably by both these forces. This may be seen in my Fig. 1 above, where the net reaction of the leeward and windward columns (for wind and dead loads together, the wind acting horizontally) are 3,800 lbs. and 47,400 lbs., respectively, acting upward in both cases. For approximately the same conditions Mr. Dunham shows corresponding reactions of 33,800 lbs. and 27,800 lbs.

Respectfully,

Chas. Worthington.

Pittsburg, Pa., Oct. 10, 1904.

Sir: Referring to Mr. Dunham's letter on the above subject in Engineering News of Oct. 6, 1904, criticising my analysis for wind-stresses in steel frame mill buildings, Mr. Dunham is not very specific, but his objection, as I infer from his illustration, is in reference to the main feature of the analysis, which is the recognition of the effect of the leverage action of the columns on the stresses in roof trusses, knee-braces and columns. I hardly see, however, how this point can be questioned, and I may possibly have misinterpreted Mr. Dunham's views, which he does not make entirely clear, either in his text or in the analysis which he presents. Notwithstanding the objections made, I consider my analysis correct in principle and along the right lines and in accordance with good, if not "accepted practice," and in so far as assumptions may depart from these general lines, the results, in my opinion, depart from the truth. This should be understood to refer to the fundamental principles and not to assumptions made relative to minor features which are not represented to be and do not need to be rigidly correct.

The assumptions referred to are as follows: In Case 1, it is assumed that the columns are hinged or free to rock at the base, while in Case 2 it is assumed that the columns present the case of a continuous beam with level supports and fixed at one end (the base), supported at the other (the top), and loaded by the knee-brace. It is evident that these conditions are not strictly realized and it is equally evident that the exact conditions cannot be determined. The true condition ordinarily falls between the two assumptions as to fixedness of column base. Each of the three assumptions affects the position of the point of contraflexure in the column.

A rigidly correct determination of the point of contraflexure for ideal conditions, involving differential equations, is given in Johnson's "The Theory and Practice of Modern Framed Structures," Art. 412, and it may be well to note here some of the deductions there given and to make a few remarks relative to the location of this point, corresponding to varying conditions.

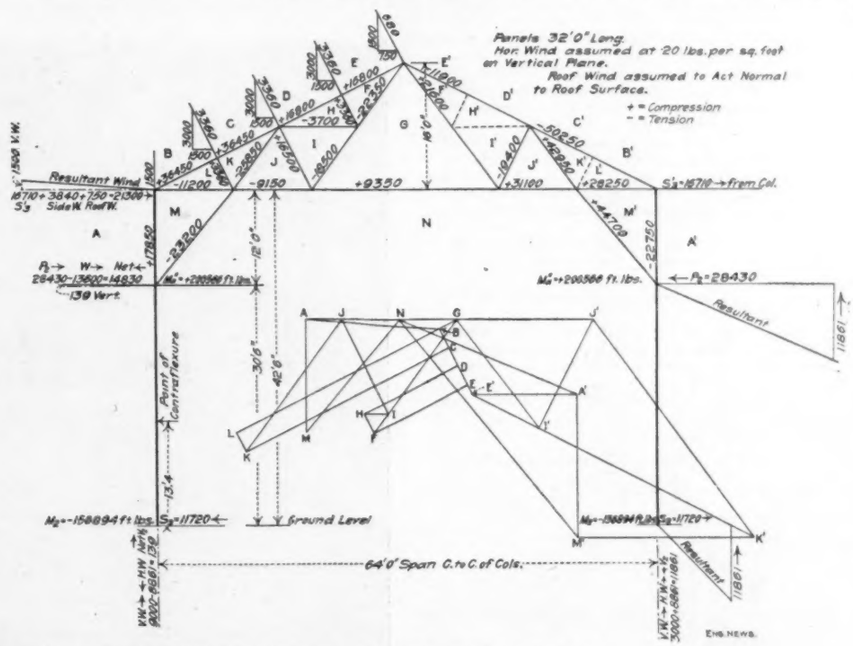
As shown in Article 151 of the work above referred to, the point of contraflexure in knee-braced columns fixed at the base occurs a distance from base of column equal to one-half to five-eighths of the distance between base of column and foot of knee-brace, the exact location depending upon the depth of knee-brace. It is evident that for columns partially fixed at the base the point of contraflexure would assume a lower position, reaching the base when the column is entirely free to rock. The designer must, of course, use his judgment as to the degree of fixedness which will be realized in individual cases. The assumption of "level supports" gives, for the depth of knee-brace assumed in the following analysis, the point of contraflexure at a distance of

$$17.1 \text{ ft.} = \left(\frac{M_{11}}{S_2} = \frac{200{,}566}{11{,}720}\right)$$

below foot of knee-brace, or 13.4 ft. above the base of column, or .44 of the distance between the base of column and foot of knee-brace; this is equivalent to the assumption of partially fixed column base, or the conditions ordinarily realized.

With the above characteristics in mind relative to the location of the point of contraflexure, if a short-cut be preferred to using the continuous girder formulas, logical assumptions which will give approximately correct results can be intelligently made relative to the location of the point of contraflexure, to correspond with the various degrees of fixedness of column base and depth of knee-brace.

As the statement and solution of the continuous girder formulas require but a few moment's time and especially in view of the above, Mr. Dunham's assumptions seem hardly necessary, for while not greatly simplifying the process they give stresses which not only materially differ from, but are opposite in character to those which I believe would actually occur, as shown in the following


Panels 32'0" Long.
Hor. Wind assumed at 20 lbs. per sq. foot on Vertical Plane.
Roof Wind assumed to Act Normal to Roof Surface.
+ = Compression
- = Tension
Resultant Wind
Side W. Roof W.
Ground Level
Point of Contraflexure
64'0" Span C. to C. of Cols.
Resultant
ENG. NEWS.

WIND STRESSES IN KNEE-BRACED MILL BUILDINGS.—BY G. H. HUTCHINSON.

tabulated comparison of wind-stresses obtained by Mr. Dunham's method and by mine:

Comparison of Stresses for Dunham's and Hutchinson's Analyses.

Member.	Dead and Wind = Total. Dunham.	Hutchinson.	Dead.	Wind. Dunham.	Wind. Hutchinson.
BL	+97,200	+94,200	+50,000	+47,200	+44,200
CK	+93,000	+91,200	+47,000	+46,000	+44,200
DH	+75,200	+65,000	+47,000	+28,200	+18,000
EF	+69,800	+66,600	+43,000	+26,800	+23,600
FG	−41,400	−53,200	−24,000	−17,400	−29,200
FH	+10,800	+13,600	+9,000	+1,800	+4,600
GI	−29,200	−38,800	−15,000	−14,200	−23,800
HI	−12,000	−15,000	−10,000	−2,000	−5,000
IJ	+26,000	+34,300	+13,000	+13,000	+21,300
JK	−27,400	−41,600	−7,000	−20,400	−34,600
KL	+8,600	+9,500	+6,000	+2,600	+3,500
LM	−87,000	−60,000	−45,000	−42,000	−15,000
JN	−78,600	−51,200	−38,000	−40,600	−13,200
GN	−49,400	−13,500	−24,000	−25,400	+10,500
MN	−18,600	−32,300	.....	−18,600	−32,300
B^1L^1	+55,000	−9,300	+50,000	+5,000	−59,300
C^1K^1	+52,200	−12,300	+47,000	+5,200	−59,300
D^1H^1	+68,400	+37,400	+47,000	+21,400	−9,600
E^1F^1	+64,000	+32,000	+43,000	+21,000	−11,000
F^1G	−33,200	−1,900	−24,000	−9 300	+22,100
F^1H^1	+9,000	+11,500	+9,000	.....	+2,500
G^1I^1	−22,400	+9 800	−15,000	− 8,400	+24,800
I^1J^1	+5,000	−9,200	+13,000	−8,000	−22,200
J^1K^1	+11,400	+45,500	−7,000	+18,400	+52,500
K^1L^1	+5,600	+6,000	+6,000	.....	.....
L^1M^1	−74,000	−13,000	−45,000	−29,000	+32,000
J^1N	−65,600	−2,700	−38,000	−27,600	+35,300
M^1N	+18,600	+54,800	.....	+18,600	+54,800
AM	.....	+21 200	.....	.....	+21,200
A^1M^1	.....	−26,500	.....	.....	−26,500
H^1I^1	.....	−13,000	−10,000	.....	−3,000

I realize that the analyses for the several cases as given in "Theory and Practice of Modern Framed Structures" are presented in a more complex manner than is desirable, owing to the results for all the cases having been grouped for comparison for purpose of presentation before an Engineers' Society, making the process for a single case appear more involved than it really is. On this account I give below, upon the request of parties interested in the application of this analysis, an analysis for a building of the same dimensions as assumed in my paper, but without a ventilator and for wind normal to roof, corresponding to Case 5.

WIND ANALYSIS.—Case V., without Ventilator.—See Articles 490-492, in "Modern Framed Structures" for nomenclature.

To find S_2 = Horizontal shear or reaction at foot of each column of bent.

The wind is considered concentrated at foot of column, at foot of knee-brace, at top of column and at panel points of roof. The concentration at foot of column is disregarded in the analysis of stresses in trusses and columns, but considered in calculation of foundations and anchor bolts.

It is evident that

$$\begin{cases} (6 + 15.25)\,32 \times 20 = 13{,}600 \text{ concentration at foot of knee brace.} \\ 6 \times 32 \times 20 = 3{,}840 \text{ concentration at top of column, from side of building.} \\ 1{,}500 \times 4 = 6{,}000 \text{ roof truss concentration.} \end{cases}$$

$$23{,}400 \div 2 = 11{,}720 = S_2.$$

$$k = \frac{a}{l} = \frac{30.5}{42.5} = .7176.$$

Eq. (9a) $$P_2 = \frac{2\,S_2}{2k - 3k^2 + k^3 + 2(1-k)} = 28{,}430.$$

Eq. (6) $$M_2 = \frac{-P_2 l}{2}[2k - 3k^2 + k^3] = -156{,}894 \text{ ft.-lbs.}$$

Eq. (10) $$S^1_2 = \frac{-P_2}{2}[2k - 3k^2 + k^3] + P_2 k = 16{,}710.$$

Eq. (8) $$M^{11}_x = (-M_2) + S_2 a = 200{,}566.$$

m = mom. of hor. forces about base
$= 13{,}600 \times 30\tfrac{1}{2} + 3.840 \times 42\tfrac{1}{2} + 6{,}000 \times 50\tfrac{1}{2} = 881{,}000.$
for wind on side of bldg. and hor. comp. on roof.

$$V_2 = \begin{Bmatrix} \text{Vertical reactions at columns due to wind on side of building and roof for } x = o. \end{Bmatrix} = \pm \begin{Bmatrix} \text{Vertical reactions due to wind on side of building and to horiz. comp. on roof.} \end{Bmatrix} + \begin{Bmatrix} \text{Reactions due to vertical component of wind on roof.} \end{Bmatrix}$$

$$\text{Vertical reaction due to wind on side of building and horiz. comp. on roof.} = \pm \frac{m}{b} + \frac{2\,S^1_2 l - 2\,P_2 a}{b} = \frac{567{,}120}{64} = \pm 8{,}861.$$

$$\text{Vertical reaction at leeward column from vertical comp. of wind on roof.} = \frac{3{,}000(8 + 16 + 24) + 1{,}500 \times 32}{64} = +3{,}000$$

Ditto at windward column $= 12{,}000 - 3{,}000 = +9{,}000.$

Hence,

$$V_2 \text{ for } x = 0 = \begin{Bmatrix} \text{Total vertical reaction at leeward column.} \end{Bmatrix} = +8{,}861 + 3{,}000 = +11{,}861.$$

$$V_2 \text{ for } x = 0 = \begin{Bmatrix} \text{Total vertical reaction at windward column.} \end{Bmatrix} = -8{,}861 + 9{,}000 = +139.$$

Practically all the mathematical expressions required for the determination of the various loads, reactions, sheare and moments are given. The graphical stress diagram and the stress sheet are also given. The normal wind pressure per square foot of roof surface for various pitches corresponding to a given pressure per square foot on a vertical plane can be determined from Hutton's formula or more readily from tabulated statements given in standard books of reference.

Very truly yours, G. H. Hutchinson.

FRENCH QUADRUPLE-EXPANSION ENGINE OF 1,500 HP. AT THE ST. LOUIS EXHIBITION.

The power plant of the St. Louis Exhibition includes two interesting vertical high-speed engines. One of these is a three-cylinder triple expansion, 1,000 HP. engine of the well-known Willans central-valve type, built by J. A. Bradly, Jr., & Co., of Pittsburg, Pa.; this runs at 277 revolutions per minute. The other is a quadruple-expansion six-cylinder engine of 1,500 HP., running at 335 revolutions. This was built by the Societe Delaunay Belleville, of St. Denis, France, and is the subject of the present article. A general view is shown in Fig. 1.

The engine has three pairs of tandem cylinders, side by side: the three upper cylinders are 13.6 ins. diameter, and the three lower cylinders are 27.2 ins. diameter; all are of 18.8 ins. stroke. The arrangement is shown in the longitudinal section Fig. 2. The middle cylinder, A, of the upper row, is the high pressure cylinder, and from it the steam is carried to the two adjacent cylinders, B and C, which are the first intermediates. The steam is then carried to a reheater, and thence to the middle cylinder, D, of the lower row, which is the second intermediate. From this the steam is carried to the two low-pressure cylinders E and F, from which it is finally conducted to the condenser. The upper and lower rows of cylinders are separated by sleeve castings which also form the cylinder covers. The pistons are of cast steel.

The connecting rods are attached to a crank shaft whose three cranks are set at 120°; one end of this shaft is directly connected to the shaft of the circulating and air pumps of the condenser, while the other end is directly connected to the generator shaft. The crank shaft bearings are formed in the bed plate, upon which is the frame containing the guide bars; this frame forms a casing which is fitted with air tight doors, thus entirely enclosing the working parts. On the top of the frame is a box girder casting, and upon this rest the lower cylinders, the object of the girder being to separate the cylinders from the working chamber. Access to the lower cylinders is obtained through this girder. The cylinders are lagged with insulating material covered with steel jackets, but there is no steam jacketing.

Steam can be introduced into all the cylinders to heat them before the engine is started. A special by-pass allows of admitting steam to the high-pressure cylinder without passing the throttle, in order to bring the engine to the proper speed to put the generator in synchronism with other generators feeding into the same system. The speed is regulated by a centrifugal governor, driven by a bevel gear on the crank axle. The engine occupies a floor space 12.36 × 9.18 ft., and is 17.05 ft. high. Its weight is about 40 tons.

The steam distribution of each group of cylinders is controlled by piston valves of central admission, operated by eccentrics on the crank shaft, as shown in the cross section, Fig. 3. A special feature of the engine is its lubrication under pressure, maintaining a constant supply of oil at all bearings and pins, thus making an easy-running engine and preventing wear of the parts. Two oil pumps in the bed plate are driven from eccentrics on the crank shaft, and force the oil through holes bored in the shaft and through the hollow connecting rods to the crank pins, bearings, crosshead pins, guides, etc. The engine has a surface condenser, with a centrifugal circulating pump and an air pump of the Edwards type, with cylinders 14 × 7 ins.; both are driven from one shaft, which is coupled directly to the engine shaft.

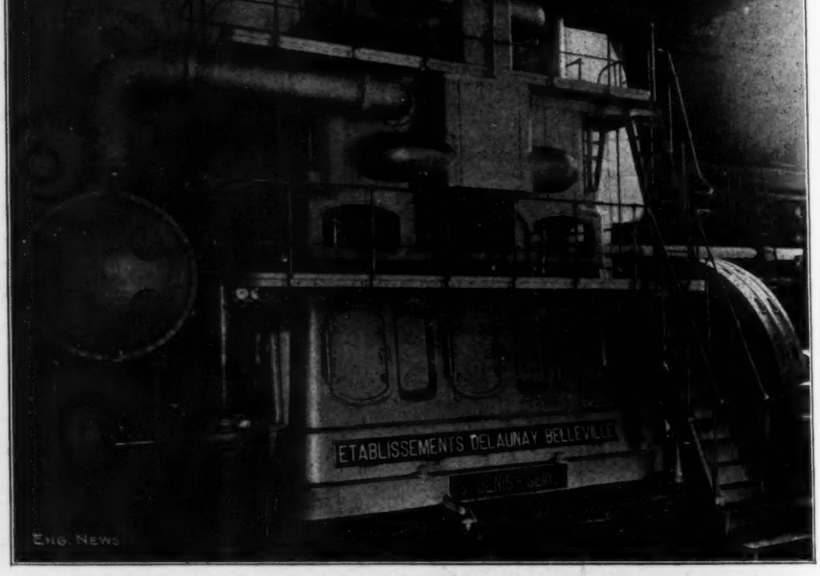


FIG. 1. 1,500-HP. QUADRUPLE-EXPANSION SIX-CYLINDER ENGINE AT THE ST. LOUIS EXHIBITION.
Societe Delaunay-Belleville, St. Denis, France, Builders.

As already noted, the other end of the engine shaft is coupled to the shaft of the generator, which is an alternating-current machine of 1,000 KW., with its exciter unit mounted on an extension of the shaft. The generator and exciter were built by the Societe l'Eclairage Electrique, of Paris.

THE PREVENTION OF LEAD POISONING FROM WATER SERVICE PIPES AT TEPLITZ-SCHÖNAU, BOHEMIA.

Trouble from lead in city water was experienced a year ago at Teplitz-Schönau, in Bohemia. The city supply is fed from two reservoirs, one drawing water from an underground supply, the other from mountain springs. The latter water is much lower in hardness than the other, and it was this softer water which gave rise to complaints of high lead-content. The highest permissible maximum of lead is placed at 1 milligram per liter (1 part per 1,000,000), but some samples of water showed 4½ times that amount.

The house service pipes in the city are universally made of "sulphurized" lead pipe; that is, lead pipe on the interior of which a thin film of lead sulphide has been formed by treatment with hydrogen sulphide. Such pipe has been thought by the Austrian authorities to be immune against the dissolving action of water. The Austrian Ministry of the Interior in 1884 promulgated a set of rules for water-supply constructions, which specify, as regards the house service pipes, that such pipes shall be: Cast or wrought iron, but not galvanized iron; or, tin-lined or sulphurized lead pipes, but the sulphurized pipes shall be used only when a test shows that the water in question does not attack such pipes. Since then the use of sulphurized lead pipes has become almost universal in Austria.

The observations in the present case showed that the sulphurized pipes were materially affected by the water in use, at any rate by the soft spring water. The analyses of the two waters in question are:

	Parts per 1,000,000 — Ground water.	Spring water.
Silicic acid	.....	8.2
Iron oxide and alumina	trace	3.3
Lime	43.8	13.7
Magnesia	9.7	1.9
Sulphuric acid	45.3	23.3
Chlorine	14.2	5.0
Ammonia	.....	trace
Nitric acid	trace	trace

As above stated, it was in all cases the spring water, which is the lower in hardness, which showed the high lead. Moreover, the highest lead was found in samples taken on Monday from the tap of a public school, the water having stood in the service pipe for some 36 hours. An investigation of the subject was then made by Prof. Wm. Ginti, who reported that the dissolving action upon the lead pipe was directly traceable to the low hardness and relative alkalinity of the spring water. He reported that the lead sulphide lining is not a complete protection, but merely retards the dissolving action. To prevent further trouble he advised that a plant be built for increasing the hardness of the spring water by slow passage through a bed of crushed limestone and gypsum, the water being previously charged with carbonic acid at the rate of 28 grams per cu. m. (or 28 parts per 1,000,000) of water.

Such a plant is now being built. It comprises two duplicate vertical cylinders about 3 ft. in diameter by 13 ft. high, filled with crushed limestone and gypsum in alternate layers; the supply pipe enters at the bottom, carbonic acid being admitted to this pipe by a branch. The treated water flows from the top of the cylinder into a small settling tank, going thence into the distributing main. Two treating cylinders are provided, to offer facilities for cleaning and repairing without interrupting the treatment. As a preliminary precaution, before this plant is ready for use, it is recommended that users let a quantity of water flow to waste each morning before drawing water for domestic use, thus getting rid of the water that stood over night in the service pipe. The case is reported in "Z. d. Oesterr. Ing. u. Arch. Ver.," of Oct. 21, 1904.

In our issues of Jan. 4, 1900, and Oct. 9, 1902, we abstracted accounts of investigations of the action of water on lead and other kinds of service pipe, made by the Massachusetts State Board of Health in 1897 and 1900, inclusive, and published in the annual reports of the Board for 1898 and 1900. These investigations were prompted by cases of lead poisoning in Massachusetts, and showed very high lead contents in some of the waters of that state.

Some of the conclusions of Mr. H. W. Clark, Chemist of the Board, were: (1) That "a potable water in Massachusetts to have any dangerous lead-dissolving action must contain considerable free carbonic acid"; (2) that "the greater the hardness of the water as compared with its free carbonic acid, the less effect did this carbonic acid have upon lead"; (3) that while the exact quantity of lead that may prove harmful is unknown and may vary with different people "it is known that the continuous use of water containing quantities of lead as small as 0.05 parts per 100,000, or about 1-33 grain per gallon has caused serious injury to health"; (4) that "the use of lead service pipes, especially in connection with ground water supplies containing considerable free carbonic acid, should be avoided wherever possible," and that cement-lined iron, tin or tin-lined lead pipe should be substituted. If tin-lined pipe is used the tin should have "a considerable thickness." Brass pipe appeared to be little acted upon by water, "but copper salts are considered to be more harmful than salts of tin."

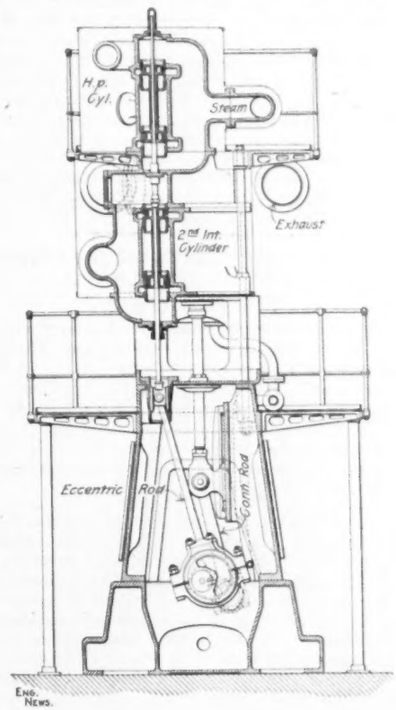


Fig. 3. Cross-Section of Quadruple Expansion Engine.

In contrast with one of the above conclusions, the dictum of Professor Ginti, who investigated the Teplitz-Schönau case, should be noted: that lead-poisoning of water is rendered possible by the absence of bi-carbonates of calcium and magnesium, and is directly caused by the dissolved air; that the dissolved air forms a hydroxide of lead, which is slightly soluble in water, but which is precipitated by the before-mentioned bicarbonates; that the presence of chlorides or nitrates in considerable amounts may cause lead-poisoning of the water even in presence of bi-carbonates; that the attack of the lead by the water is more rapid when the pipes are sometimes empty and thus expose the lead to moist air, which acts more rapidly than the air contained in the water; and, finally, that when the "incrusting solids" in

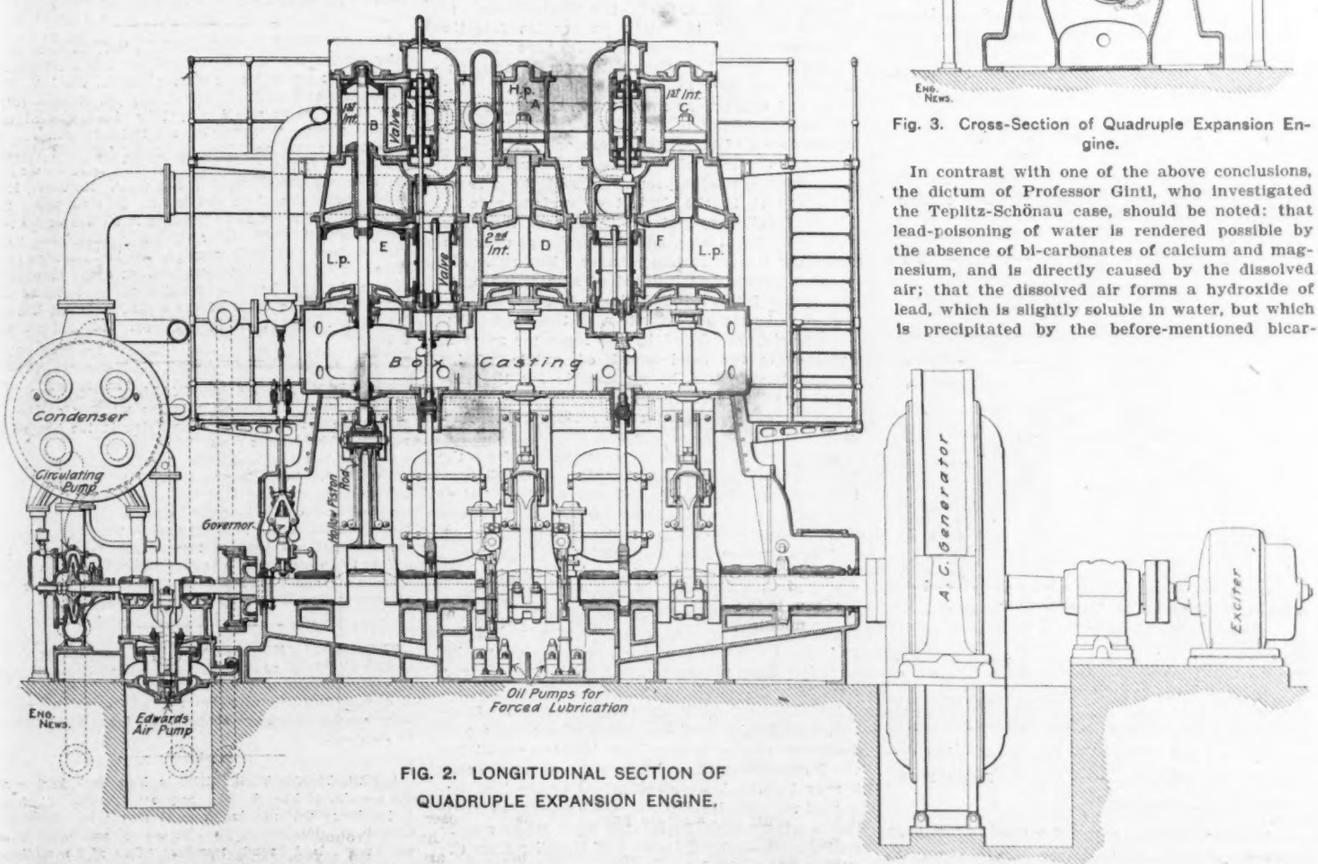


FIG. 2. LONGITUDINAL SECTION OF QUADRUPLE EXPANSION ENGINE.

the water form a complete coating over the inside of the pipe, the lead is completely protected against being attacked by the water. The presence of carbonic acid gas in the water is not considered by him to promote solution of the lead; as above reported, he recommends the addition of this substance in the treating apparatus, but this is for the purpose of promoting the solution of calcium and magnesium in the form of bi-carbonates. In the Teplitz-Schönau case it was considered impracticable to supply bi-carbonates in the form of sodium bi-carbonate, for the reason that the hard ground water which forms part of the supply is high in sulphates, so that, in mingling with the softer water treated with sodium bi-carbonate, precipitation and resulting turbidity might be caused. The greater cost of treatment with sodium bi-carbonate is also an argument against its use.

EXPERIMENTS AND FORMULA FOR THE EFFICIENCY OF TANGENTIAL WATER-WHEELS.

By B. F. Groat.*

In coordinating a series of laboratory experiments on impulse water-wheels, the efficiency function was chosen in the form

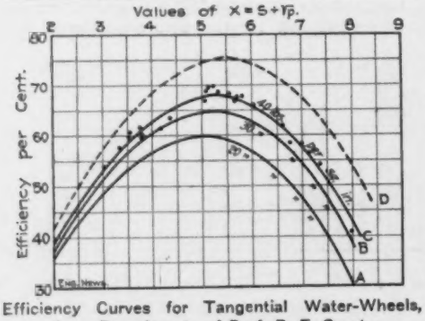
$$e_s = a\,x^3 + b\,x^2 + \frac{c}{p}\,x + d\,x$$

where $x = \frac{s}{\sqrt{p}}$, s = speed in r. p. m. under a pressure of p lbs. per sq. in. a short distance back of the nozzle. About 110 observations were made, by several inexperienced students, of the brake efficiencies of a 2-ft. impulse turbine (or tangent wheel) at various speeds under different heads. About 20 of the observations were thrown out, being manifestly in error and time not being taken to correct them. The remaining 90 observations were reduced by least squares, the normal equations being formed from

$$\frac{e_s}{x} = a\,x^2 + b\,x + \frac{c}{p} + d,$$

e, x, and p being the measured quantities. This reduction gave the following for the efficiency function:

$$e_s = -.00000116x^3 - .00119x^2 - \frac{.582}{p}\,x + 2.37x.$$



Efficiency Curves for Tangential Water-Wheels, from Experiments of Prof. B. F. Groat.

The plotted points on the diagram represent observed efficiencies for values of p between 38 and 42 lbs. per sq. in. The other observations are not plotted. The efficiency curves obtained by drawing free-hand curves through the observations taken under 20, 30 and 40 lbs. pressure practically coincide with the corresponding curves shown above.

Curves A, B and C are plotted from the formula $e = -.00000116\,x^3 - .00119\,x^2 - \frac{.582}{p}\,x + 2.37\,x$ for p = 20 lbs., 30 lbs., 40 lbs., respectively.

e = brake efficiency, $x = \frac{s}{\sqrt{p}}$, s = speed in r. p. m., p = pressure of lbs. per sq. in., a short distance back of the nozzle.

Curve D is plotted from the same formula, but having the third term omitted.

Upon computing e for the observed values of x and p, the majority of results were found to agree to within 2 per cent. of the observed efficiency. The probable error was probably much less than 2 per cent. With more accurate measurements and experienced observers, a function of the form employed may give even closer approximations.

A plot of the loci of the function for p = 20, 30 and 40 lbs. per sq. in. is attached hereto, as is also a plot of the function formed by omitting the term $-\frac{.582}{p}\,x$ (dotted lines). The latter curve is of interest as it seems to be somewhere near the hydraulic efficiency of the motor.

The above-described experiments and formula may be useful for checking laboratory efficiency observations. The function seems to be pretty close to the right form, at least for the wheel experimented upon.

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PROTECTION OF PUBLIC WATER SUPPLIES IN OHIO.

Ohio is rapidly coming to the front as a State that protects the purity of its public water supplies. For a number of years past all projects for new water supplies and for the disposal of sewage have been subject to the approval of the State Board of Health. Frequently the board has withheld its approval of plans because they did not provide for water or sewage purification, as the case might be, or has approved plans subject to orders at its own option for purification works. As examples of the attitude of the board, the following review* of its action on plans for water supply and sewerage at its regular meeting on June 22, 1904, may be given.

A proposed new surface water supply for Barnesville, from a drainage area of 600 to 700 acres, was approved, subject to two conditions: (1) That the local authorities enforce certain rules and regulations for the sanitary protection of the drainage area, as drawn by the State Board of Health, and (2) the State board reserved to itself the right to require filtration "whenever in its opinion" the rules "are not being properly observed or when the water becomes sufficiently polluted to warrant filtration." The rules and regulations embodied in the approval of the plans prohibit in detail the construction and maintenance of privies and cesspools, stables, pig stys and the like within certain distances of the proposed impounding reservoir and the streams tributary to it, the deposit of various specified kinds of polluting material in or near the reservoir or its tributaries, and the discharge of manufacturing wastes in like manner. The rules also provide for the appointment of a local inspector, who must inspect the drainage area every two weeks and note any violation of the rules. Any case of sickness found by the inspector must be reported to the local board of public affairs, "which board shall be responsible for the care of the patient with respect to his causing any pollution of the water of the Barnesville Reservoir."

In approving plans for a water supply from the north branch of the Licking River for Newark the board required the installation of filters before the water was offered for consumption. Plans for mechanical filters at Marietta were approved, and also for a supply from wells for Zanesville.

Sewerage plans for a district of Lorain, now being privately developed, were approved, with certain conditions as to the early construction of purification works. Sewerage systems for Delphos and for Urichsville, without purification works, and for Newark, apparently with such works, were also approved.

Dr. C. O. Probst is secretary of the Ohio State Board of Health, and on receipt of postage would doubtless send a copy of the bulletin containing the Barnesville rules. Those rules, or such rules, it may be added, were authorized by the last session of the Ohio legislature, which gave municipalities sanitary jurisdiction over their water supply drainage areas for a distance of 20 miles from the city or village limits.

*From the "Ohio Sanitary Bulletin," for July and August, 1904, which gives the action in more detail.

GRIT CHAMBERS, SETTLING TANKS, SINGLE CONTACT beds and storm water filters are proposed at Sheffield, England, as a substitute for the chemical precipitation works built 18 years ago. The plans provide for 16 settling tanks with a holding capacity of 18,000,000 U. S. gallons, 60 contact beds of one-half acre each, and 16 one-acre, storm water filters. The estimated cost, including various accessories, but excluding land, is £270,000 or $1,310,000. The site of 100 acres adjoins the present works and is already owned by the city. Presumably, the old chemical precipitation tanks will be utilized in the new scheme. If found necessary, second contact beds will be installed after the works as now planned have been given a practical trial. The plans as outlined must be approved by the Local Government Board before they can be carried out. The scheme is based on the conclusions drawn from experimental works, if these can be so-called, treating 1,200,000 U. S. gallons of sewerage a day in 1903-4. Mr. Charles P. Wike, M. Inst. C. E., City Surveyor of Sheffield, has directed the investigations. Although the studies included septic tank effluents Mr. Wike concluded that the contact beds would give satisfactory results if the applied sewage had not attained a marked degree of septic action; according to he recommended settling in place of septic tanks.

THE DROP-TESTING MACHINE of the Master Car Builders' Association, designed for testing couplers, has been removed from Altoona to the laboratory of Purdue University, at Lafayette, Ind. Its drop weighs 1,600 lbs., and is of forged steel. Its weight meets the requirements of the Master Car Builders' specifications for coupler testing as well as those of the International Association for Testing Materials with reference to axle testing. It can be made to meet the requirements of the last named association for rails by applying disks to a cylindrical opening in the hammer which serve to increase its weight to 2,000 lbs. The drop is handled by a wire cable served by a reversible hoisting engine within the locomotive laboratory. The anvil or base of the machine weighing 17,000 lbs. is carried by a nest of coiled springs which in turn are mounted upon a suitable foundation plate. It is assumed that the spring support of the anvil makes the machine easily reproducible; that is, that the resisting qualities of a similar anvil, similarly supported, will be the same whether the machine rests upon a rock foundation or upon soft soil. The machine rises to a total height of 55 feet.

AN EXPLOSION OF DYNAMITE killed one man, mortally injured two others, and more or less seriously injured 50 people in Mount Vernon, N. Y., Nov. 2. The dynamite was temporarily stored in a cut under a highway bridge which was destroyed by the explosion. The number of cases that exploded are variously estimated at from two to ten; and, judging by the fact that the explosion was felt in a radius of ten miles, the amount of explosive was nearer ten than two cases. Moreover, it is stated that a deep hole was left in the rock where the dynamite shed had stood. Windows in all houses for two blocks around were shattered, many chimneys were thrown down and walls cracked, causing damages estimated at $50,000. That more men were not killed was due to the fact that the explosion occurred during the lunch hour. The cause of the explosion is not known, but is attributed to a lighted cigar falling from the bridge.

THE FAILURE OF A SMALL RESERVOIR AT Winston-Salem, N. C., early in the morning of Nov. 2 caused the death of nine people living in the immediate vicinity. The reservoir was 62 ft. square on the inside and was formed by brick walls 35 ft. in height above the bottom. The reservoir was built in 1881 or 1882 by a private company. In 1894 the city bought the waterworks and added 10 ft. to the height of the walls, at the same time increasing their thickness. The 6 ft. added to the thickness was built on the original reservoir bottom and at the same time the bottom lining was increased by a depth of 12 ins. There seems to have been little or no attempt to bond the old and new walls together.

THE LEWIS AND CLARK EXHIBITION, at Portland, Ore., which is to open on June 1, and continue till Oct. 15, 1905, is now allotting space to exhibitors. About $5,000,000 is to be expended on the Exposition, and a large number of exhibits shown at St. Louis will be taken to Portland.

A FERRYBOAT WAS SUNK in New York harbor on the morning of Nov. 4. The ferryboat "Columbia," of the Union Ferry Co., running from Montague St., Brooklyn, to Wall St., Manhattan, New York City, was rammed by the Long Island Sound steamboat "City of Lowell." A dense fog hung over the river at the time. All of the many people on the ferryboat were rescued.

STATE PRISON FOR CAUSING A COLLISION was the sentence given to a locomotive engineer in Mexico a few days ago. The collision occurred last spring and resulted in a large number of deaths. The engineer of one of the trains was found guilty of criminal carelessness and was sentenced to four years in prison.